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Sensitivity analysis of rigid pavement design inputs using mechanistic-empirical pavement design guide

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Sensitivity analysis of rigid pavement design inputs using mechanistic-empirical pavement design guide

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Signatures have been redacted for privacy

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ABSTRACT

Pavement design procedures, available in the literature, do not fully take advantage of mechanistic concepts, which make them heavily rely on empirical approaches. Because of the heavy dependence on empirical procedures, the existing design methodologies do not capture the actual behavior of Portland cement concrete (PCC) pavements. However, reliance on empirical solutions can be reduced by introducing mechanistic–empirical methods, which is now adopted in the newly released mechanistic-empirical pavement design guide (MEPDG). This new design procedure incorporates a wide range of input parameters associated with the mechanics of rigid pavements. To compare the sensitivity of these various input parameters on the performance of concrete pavements, two jointed plain concrete pavement (JPCP) sites were selected in Iowa. These two sections are also part of the Long Term Pavement Performance (LTPP) program where a long history of pavement performance data exists. Data obtained from the Iowa Department of Transportation (Iowa DOT) Pavement Management Information System (PMIS) and LTPP database were used to form two standard pavement sections for the comprehensive sensitivity analyses. The sensitivity analyses were conducted using the MEPDG software to study the effects of design input parameters on pavement performance of faulting, transverse cracking, and smoothness. Based on the sensitivity results, ranking of the rigid pavement input parameters were established and categorized from most sensitive to insensitive to help pavement design engineers to identify the level of importance of each input parameter. The curl/warp effective temperature difference (built-in curling and warping of the slabs) and PCC thermal properties are found to be the most sensitive input parameters. Based on the comprehensive sensitivity analyses, the idea of developing an expert system was introduced to help the pavement design engineers identify the input parameters that they can modify to satisfy the predetermined pavement performance criteria. Predicted pavement distresses using the MEPDG software for the two Iowa rigid pavement sites were compared against the measured pavement distresses obtained from the Iowa DOT's PMIS and comparison results are discussed in this study.

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Special thanks are due to my family, for their love, patience, and support. The last but not least, I would like to thank my friends for their encouragement, suggestions, and support.

CHAPTER 1

INTRODUCTION

1.1 Research Objective

The objective of this research was to identify the sensitivity of input parameters needed for designing the jointed plain concrete pavements (JPCPs) used in the newly released mechanistic-empirical pavement design guide (MEPDG) (a.k.a. NCHRP Project 1-37A Mechanistic-Empirical Pavement Design Guide for Design of New and Rehabilitated Pavement Structures). The findings of this study will guide the state department of transportations (DOTs) to determine which input parameters have either the most or the least effect on the predicted pavement distresses of transverse cracking, faulting and smoothness. In this chapter, the development of mechanistic-empirical pavement design procedures in American Association of State Highway and Transportation Officials (AASHTO) guidelines and an overview of concrete pavements is presented.

1.2 Background

Three types of concrete pavements are commonly used; (1) jointed plain concrete pavement (JPCP), (2) jointed reinforced concrete pavement (JRCP), and (3) continuously reinforced concrete pavement (CRCP).

JPCP has transverse joints spaced less than 5m apart and does not have reinforcing steel in the slab. According to a performance survey, Nussbaum and Lokken [1.1] recommended maximum joint spacings of 6m for doweled joints. JPCP can contain steel dowel bars and tie-bars across transverse joints and longitudinal joints, respectively.

JRCP has transverse joints spaced about 9-12m apart and contains steel reinforcement in the slab. Steel reinforcement in the form of wired mesh is designed to increase the structural capacity of the slab. Dowel bars and tie-bars are also used at all transverse and longitudinal joints, respectively.

CRCP does not have transverse joints and contains more steel reinforcement than JRCP. The high steel content influences the formation of the transverse cracks in close distances [1.2]. Transverse reinforcing steel is often used.

According to a 1999 survey, at least 70% of the state highway agencies in the United States used JPCP. About 20% of the states used JRCP, and about 6 or 7 state highway agencies built CRCP, most notably on high-volume, urban roadways. In this study the analysis of JPCP sections under MEPDG software was discussed.

The historical development of mechanistic-empirical (M-E) pavement design procedures in the AASHTO guides goes back to the 1986 AASHTO Design Guide. In the 1986 AASHTO guide for pavement structures, M-E design procedure was firstly defined as the calibration of mechanistic models with observations of performance, i.e. empirical correlations. It was also stated that in a multi-layered pavement system, analytic methods were the numerical calculations of the pavement responses when subjected to external loads or the effects of temperature or moisture. Then, assuming that pavements can be modeled as a multi-layered elastic or visco-elastic structure on an elastic or visco-elastic foundation, the stress, strain, or deflection could be calculated at any point within or below the pavement structure. Mechanistic procedures are referred to for the ability to translate the analytical calculations of the pavement responses to physical distress such as cracking or rutting (pavement performance). However, pavement performances are subjective to a number of factors, that cannot be exactly modeled by mechanistic methods. It is, therefore, necessary to incorporate empirical pavement performance models with mechanistic models. Thus, in the 1986 AASHTO Guide, the procedure is defined conceptually as a mechanistic-empirical pavement design procedure. [1.3]

The AASHTO pavement design guides [1.3-5] used empirical methods, which are valid for specific environmental, material, and loading conditions. In order to develop a design procedure without these limitations, the development of M-E design procedures was promoted by the AASHTO Joint Task Force on Pavements (JTFP). AASHTO JTFP recommended the research should be initiated for the later versions of the AASHTO design guides. Then, the National Cooperative Highway Research Project (NCHRP) Project 1-26 [1.6-9] was the first NCHRP project to be sponsored. After that, the second phase of NCHRP 1-26 started and was completed in 1992 with its two volumes of final reports showing the guidelines for the data input stage of the procedures [1.10]. Finally, at the conclusion of a workshop held in March 1996 in Irvine, California, JTFP concluded a long-term project for the development of a design guide based as fully as possible on mechanistic principles. This guide is titled *The NCHRP Project 1-37A mechanistic-empirical design guide for design of new and rehabilitated pavement structures* [1.11].

1.3 General Features and Scope of MEPDG

The main objective of the MEPDG was to provide a pavement design guide based on mechanistic-empirical design procedures for new and rehabilitated pavement systems, and a user-friendly software and documentation. With the help of the software, the designers would have the control to design and the flexibility to consider various features. For the design, not only were the site conditions but also the construction conditions were considered. Moreover, the MEPDG is in a format that provides the development of existing mechanistic-empirical pavement design procedures in connection with trucking, materials, construction, computers, and so on. [1.11]

1.4 Design Approach in MEPDG Design Guide

Reliability and rehabilitation design issues were updated by incorporating mechanistic approaches in relation to the 1986 and 1993 AASHTO guides and were broadened to include rehabilitation considerations not included in AASHTO guides. In the design approach, one must first consider the design inputs and analysis strategies. Design inputs are materials characterization, traffic data input, and the climate using the Enhanced Integrated Climatic Model (EICM). Next, the structural performance analysis must be considered, which is based on trial and error, beginning with standard trials obtained from agencies. Then, with initial estimates of some values, the pavement section is analyzed using the distress models. The outputs are the expected amount of distress and smoothness over time. Until satisfactory results are obtained, iterative approach continues. In summary, the following considerations are included in the MEPDG [1.11]:

- Traffic
- Climate
- Material properties (Subgrade/foundation, base, granular base)
- Existing pavement condition
- Construction factors
- Sub drainage
- Shoulder design
- Rehabilitation treatments and strategies
- New pavement and rehabilitation options
- Pavement performance (key distresses and smoothness)

- Design reliability
- Life cycle costs

Another aspect of the MEPDG is the hierarchical approach to the design inputs, which is not found in either AASHTO design guides or any other design guides. With this approach, the inputs are separated into three levels.

Level 1: Inputs provide a high level of accuracy. Level 1 inputs are used in cases of pavements with heavy traffic. These inputs require laboratory testing, field-testing (such as dynamic modulus testing of hot mix asphalt concrete), and non-destructive deflection testing. In addition, they require more tests and sources than other types.

Level 2: Inputs provide an intermediate level of accuracy, and would be considered the closest to the typical procedures applied in the AASHTO design guides. This level of inputs could be used when there is not enough equipment or testing programs. The required data are estimated through the correlations. These values could be provided from the agencies.

Level 3: Inputs provide the lowest accuracy and this level might be used for pavement with low volumes of traffic. The input values are mostly taken from the default values that are based on seasonal averages or the basic correlations.

A combination of the three input levels can also be used. However, regardless of the input level(s) used, the design procedure and the distress models are the same.

1.5 Overview of Concrete Pavement Design Methodologies

1.5.1 Empirical Pavement Design Methodologies

Empirical methods are based on experience. As more experiences were added throughout the years concerning the development of pavement thickness design, several methods have been developed by agencies. A commonly known empirical method is the AASHTO method. It is based on the results of the American Association of State Highway Officials (AASHO) road test conducted in Ottawa, Illinois, in the late 1950s and early 1960s. The first interim design guide based on the AASHO method was published in 1961 and revised in 1972 and 1981. In 1986, results of the NCHRP Project 20-7/24 recommended that the guide be expanded and revised. After the 1986 AASHTO design guide was finished, it was last revised in 1993.

After the AASHO road test, the pavement serviceability-performance concept, an outstanding feature, was developed for the thickness design. Serviceability is the ability to serve traffic in its existing conditions [1.11]. Present Serviceability Index (PSI) is one method to find serviceability condition. PSI is the condition index based on pavement roughness and distresses, such as rutting, cracking, and patching [1.11]. Designs are based on the empirical equations that are produced with PSI after the AASHO road test.

The shortcomings of empirical methods based on the AASHO road test are as follows:

- It is only valid for the same environmental, material, and loading conditions.

- Traffic values are no longer the same as those of the AASHO road test. (including axle loads and configurations, tire pressures, tire types, and volumes).
- In the road test only one type of subgrade soil is used.
- The rehabilitation of existing pavements is not addressed in the road test, and the AASHTO guide does not have a globally validated scheme for this.

1.5.2 Mechanistic-Empirical Pavement Design Methodologies

Before the new MEPDG guide was released, some industry groups [1.13-1.14] and highway agencies had already established mechanistic-empirical procedures, including Illinois [1.12]. The mechanistic-empirical design approach is a very sophisticated and reliable method of design. The complexity of the mechanistic-empirical procedure comes from use of finite element models for pavement system analysis, especially in the analysis of corners and joints on rigid pavements. Although the analyses are complex, the use of computers makes the design easier. Especially, the MEPDG's user-friendly software makes the analysis easier. Another aspect of the new MEPDG is that it does not provide a design thickness at the end of pavement analysis; instead, it provides the pavement performance throughout its design life. Therefore, MEPDG is a performance prediction tool more than an analysis tool. The design thickness can be predicted by modifying design inputs and obtaining the best performance with an iterative procedure.

The mechanistic-empirical pavement design procedure consists of inputs, structural models, pavement responses, transfer functions, and pavement distress performances as shown in Figure 1.1. Inputs for the mechanistic-empirical method are materials characterization, traffic data, and climate. Pavement materials are characterized according to their elastic properties, and it is a fact that the pavement systems have mostly non-linear properties (subgrade soil). However, since the deformations are recoverable, soil can be modeled as an elastic model under repeated application of loads [1.10].

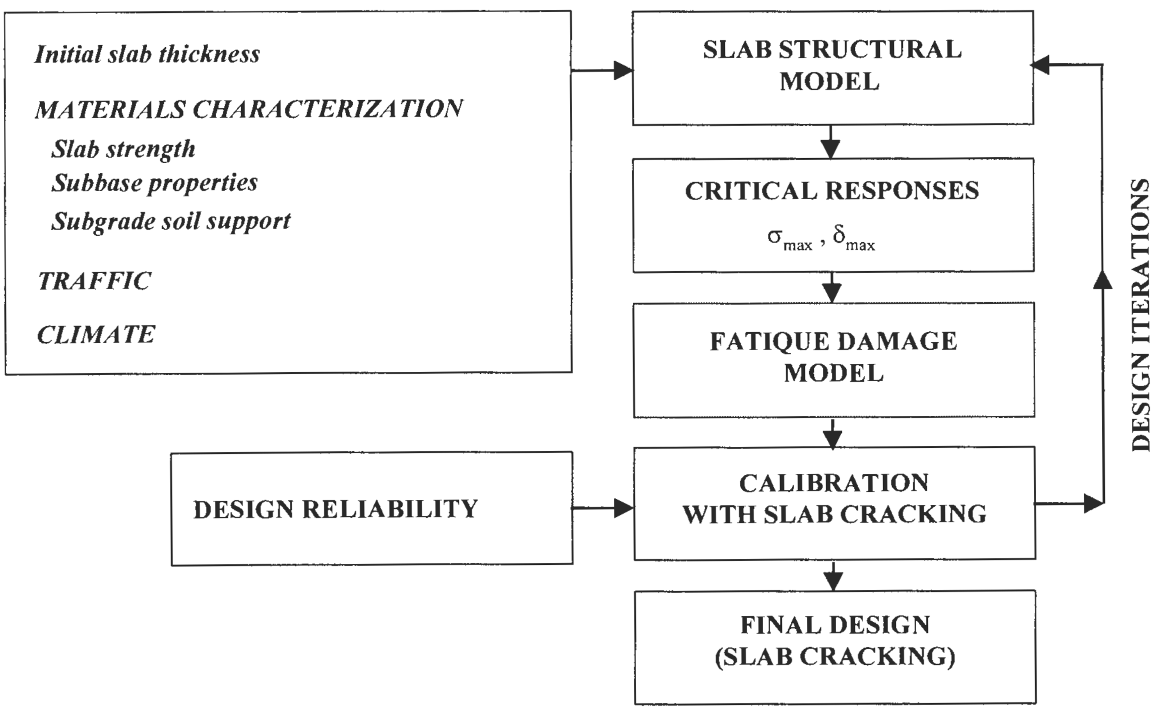

Figure 1.1 Mechanistic-empirical procedure flowchart [1.15]

For the structural modeling, the finite element models are more multipurpose and can contain stress-dependent properties (stress hardening for granular materials and stress softening for fine-grained soils). The finite element models can also include failure criteria (such as the Mohr-Coulomb model in ILLI-PAVE). Stress dependent finite element programs (such as ILLI-PAVE, MICH-PAVE, and Texas ILLI-PAVE) and elastic layer programs (such as BISAR, WESLEA, JULEA, CHEVRON, ELSYM 5, CIRCLY) are recommended for flexible pavements. [1.15]

The empirical aspect of the mechanistic-empirical pavement design process is the transfer functions. They relate the pavement responses to the pavement distress models. For instance, in MEPDG, the transfer function for the percentage of slabs with transverse cracks in a given traffic lane is used as the measure of transverse cracking, and is predicted using the following model for both bottom-up and top-down cracking [1.11]:

$$CRK = \frac{1}{1 + FD^{-1.68}}$$

Where,

CRK = predicted amount of bottom-up or top-down cracking (fraction)

FD = fatigue damage

Model Statistics:

$R^2 = 0.86$

N = 522 observations

SEE = 5.4 percent

The total amount of cracking is determined as follows:

$$TCRACK = \left(CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} \cdot CRK_{Top-down}\right) \cdot 100\%$$

where,

$TCRACK$ = total cracking (%).

$CRK_{Bottop-up}$ = predicted amount of bottom-up cracking (fraction).

$CRK_{Top-down}$ = predicted amount of top-down cracking (fraction).

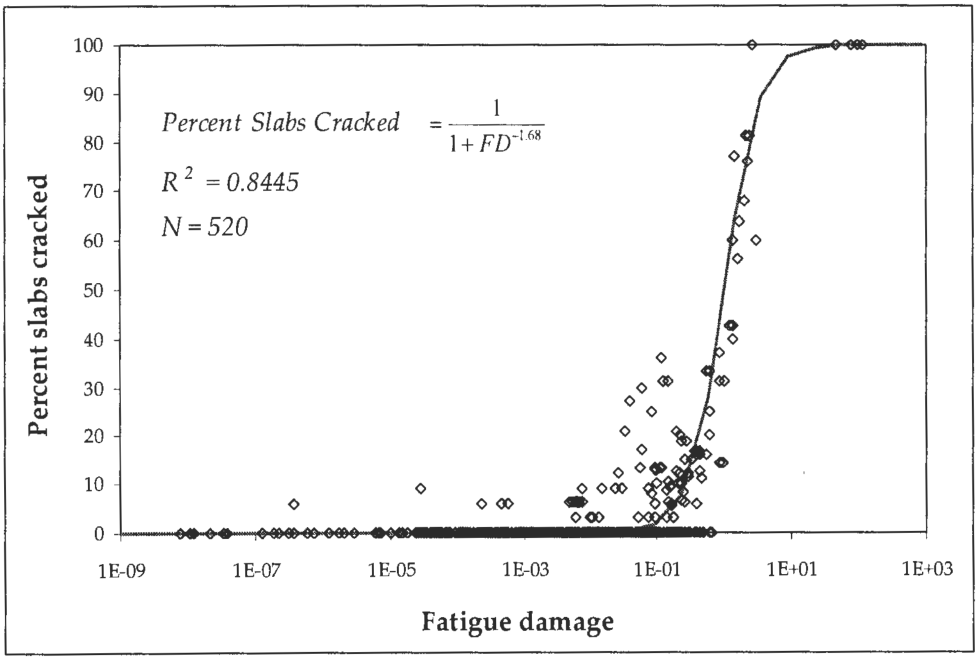

Figure 1.2 Calibration of transverse cracking based on percent slabs cracked vs. fatigue damage on 196 field sections [1.11]

The equation assumes that a slab may crack from either bottom-up or top-down, but not both. The JPCP transverse cracking model was calibrated based on performance of 196 field sections located in 24 States (see Figure 1.2). The calibration sections consist of LTPP GPS-3 and SPS-2 sections and 36 sections from the FHWA study *Performance of Concrete Pavements.*

The failure mechanism is defined as the distress of the pavement systems. In order to fail, transfer functions relate the critical responses to these failures. After relating these, an iterative design process is applied to find the thickness of the pavement.

1.5.3 Advantages and Limitations of the Mechanistic-Empirical Design Approach

The advantages of the MEPDG can be summarized as follows [1.11]:

- New loading conditions can be evaluated (such as axle configurations, damaging effects of increased loadings, high tire pressures)
- Better use of available materials can be estimated. For example, the use of stabilized materials in both rigid and flexible pavements can be simulated to predict future performances
- More reliable design (not over design or under-designed)
- Rehabilitation concept is addressed
- Seasonal effects such as thaw weakening can be included in the performance estimates

- Long-term affects can be included in the analysis
- Different sub-grades can be used to estimate performances
- Aging effects can be evaluated such as asphalt hardens with time, which, in return, affects both fatigue cracking and rutting

The limitations are below:

- Computational complexity due to structural models for pavements (such as finite element models) which requires the need of computers
- Inadequate knowledge about the design procedure
- Inexperienced personnel
- Weakness in the transfer functions

1.6 Scope of Research

Considering the current state of MEPDG, the research presented in this thesis focused on the following areas:

1. Development of sensitivity levels for inputs of rigid pavement design module of MEPDG for each pavement performance criteria using MEPDG software.
2. Development of set of recommendations for implementation plan in Iowa.

1.7 Thesis Layout

This thesis contains five chapters. Following an introduction in this chapter for concrete pavements and mechanistic-empirical design methodology, Chapter 2 provides a literature review of concrete pavement analysis methods, road tests and existing design guidelines developed for rigid pavement design.

Chapter 3 presents the design inputs used in the MEPDG with extensive review of traffic, climate, and material input parameters. Data collection, description of the sites and input parameters used in the sensitivity analyses are presented in Chapter 4. The summary of results is also presented in Chapter 4. The research findings, conclusions and recommendations are given in Chapter 5.

In the attached CD-ROM, Appendix A is located. Appendix A provides the plots for the sensitivity analyses of JPCP design inputs for each pavement performance.

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CHAPTER 2

CONCRETE PAVEMENT DESIGN METHODS AND GUIDELINES

2.1 Introduction

In this chapter, past analysis methods, tests, and procedure guidelines for concrete pavement analysis and methods that use guidelines for concrete pavement systems are reviewed. The pavement analysis methods are described under three headings: the closed-form formulas, influence charts and numerical methods (finite element methods). Along with numerical methods, the most commonly used finite element software programs for pavement design are overviewed. Afterwards the road tests are given. The pavement analysis guidelines are briefly provided.

2.2 Pavement Design Methods

Test roads, research, analytical studies, and, most importantly, the observed performance of pavements in service served as the basis for concrete pavement design practices [2.1].

The first PCC pavement was built in Bellefontaine in Ohio, 1891 by the father of PCC pavements, George Bartholomev. The first controlled evaluation of concrete pavement performance was conducted in 1909. The Public Works Department of Detroit (Michigan) conducted what was probably the first pavement test track. Based on this study, Wayne County, Michigan paved Woodward Avenue with concrete – making it the first mile of rural concrete in the United States.

Pavement design methods are based on the flexural stress and the findings of test road sections. Flexural stress is the major design factor for concrete pavements. In early road tests, such as the Bates road tests (1912 - 1923) and Pittsburg road tests (1921 - 1922), simple equations relating pavement thickness to traffic loading emerged. These were the beginnings of so-called "mechanistic-empirical" design procedures (mechanistic - based on computed pavement response; empirical - calibrated to observe pavement performance) [2.1]. As the other road tests were conducted more complex solutions were discovered and presented as influence charts for pavement design. Afterwards, with the introduction of the computer, numerical methods such as finite element methods for pavement design were developed. Thus, three methods can be used to determine the stresses and deflections in concrete pavements: closed-form formulas, influence charts, and numerical methods.

2.2.1 Closed-form Formulas

Closed-form formulas are the analytical solutions for determining the stresses and deflections of rigid pavement systems. The well-known formulas and assumptions are presented below.

2.2.1.1 Goldbeck's Formula

In 1919, Goldbeck [2.2] developed the earliest formula for use in concrete pavement design. The same equation was applied by Older [2.3] in the Bates road test. Goldbeck's assumption of the pavement system as a simple cantilever beam with a load concentrated at the corner yielded his simple equation: for a given concentrated load of P, a cross section at a distance x from the corner, the bending moment of Px and the width of section is 2x (see Figure 2.1). When the subgrade support is neglected and the slab is considered as cantilever beam, Goldbeck's equation for stresses is as follows:

$$\sigma_c = \frac{Px}{\frac{1}{6}(2x)h^2} = \frac{3P}{h^2}$$

where,

σ_c = stress due to corner loading

P = concentrated load

h = thickness

x = distance from the corner

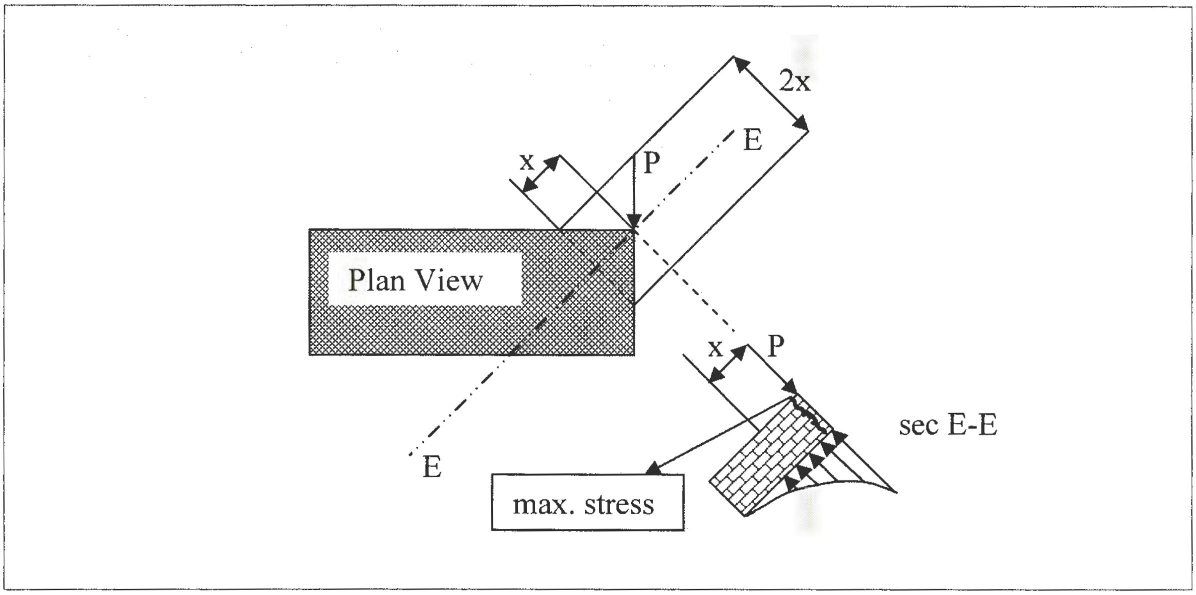

Figure 2.1 Goldbeck's formula [2.4]

2.2.1.2 Westergaard Theory

Harold Westergaard [2.5] developed closed-form analytical equations for the determination of stresses and deflections in concrete pavements. His equations can be applied only to large slabs on a Winkler (or liquid) foundation loaded with a single-wheel load with a circular, semicircular, elliptical, or semi elliptical contact area (see Figure 2.2). A Winkler foundation is characterized by a series of springs attached to the plate. Westergaard published his first equations in 1926, and published his in-depth studies and revised equations in 1927, 1929, 1933, 1939, 1943, and finally in 1948. He published new derived equations in 1948. In 1985, Ioannides et al. [2.12] demonstrated that Westergaard's several equations were erroneous, and provided the correct forms of the equations. Moreover, it was determined that the original edge stress equation (1926) was also incorrect and his later formula (1948) should be used.

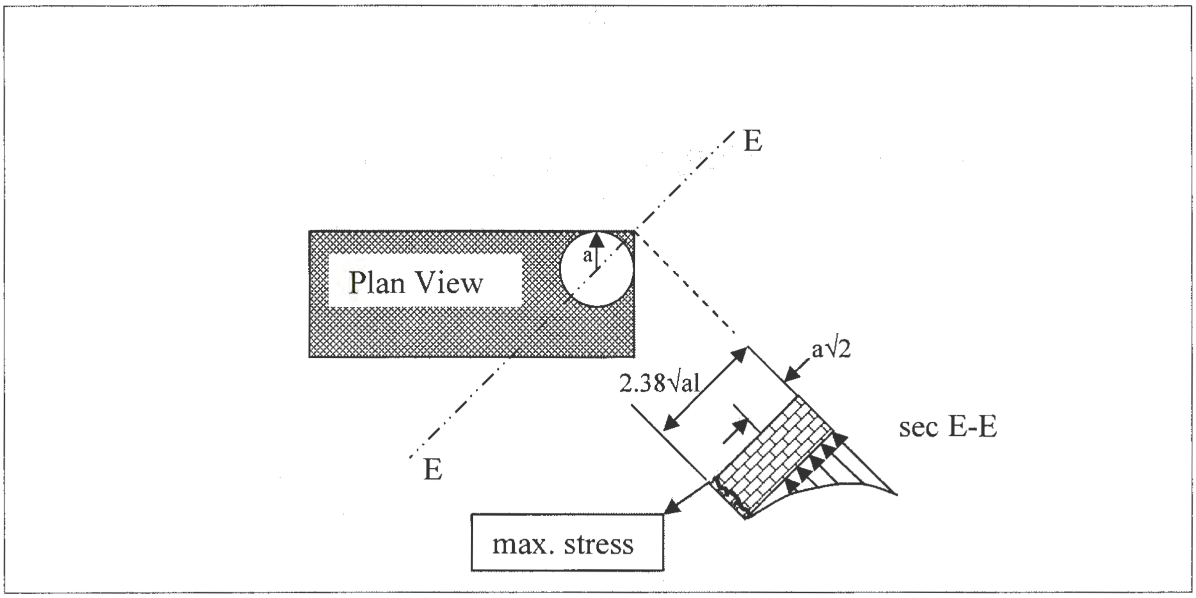

Figure 2.2 Westergaard corner loading

In his studies [2.5-11], Westergaard investigated three different loading conditions: (1) interior, (2) edge, and (3) corner (see Figure 2.3).

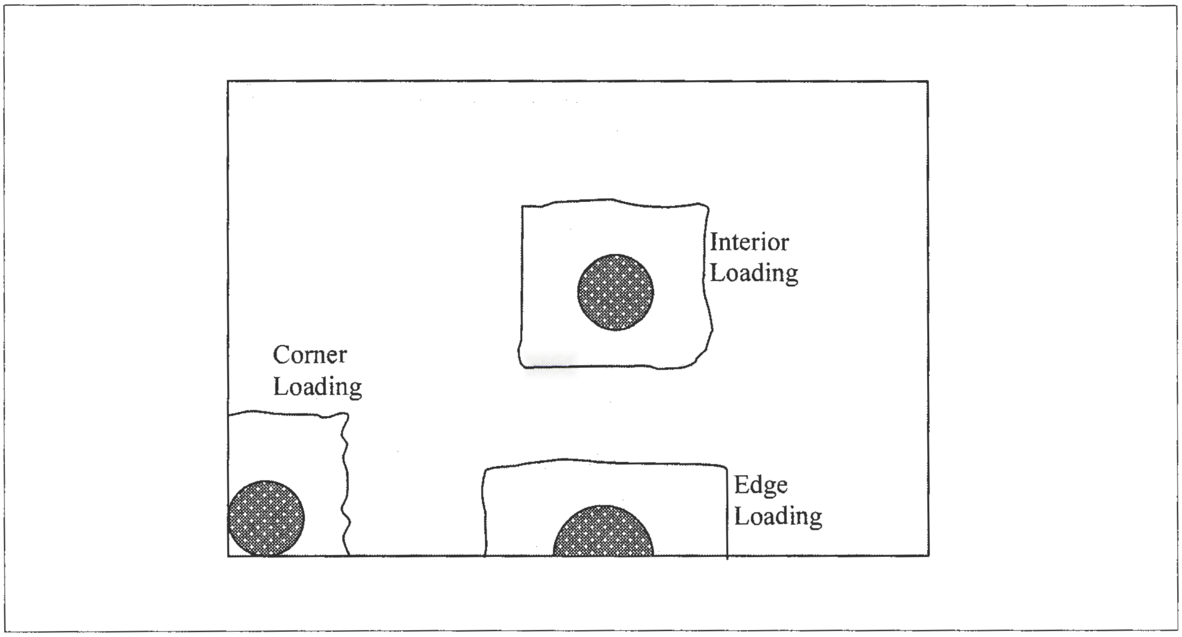

Figure 2.3 Westergaard different loading locations.

Westergaard introduced the radius of relative stiffness *(l)* which measures the stiffness of the slab relative to that of the subgrade. It is defined by the following equation:

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

l = Radius of relative stiffness, in.

E = Modulus of elasticity of the pavement, lbf/in^2.

h = Thickness of the pavement, in.

μ = Poisson's ratio.

k = Modulus of subgrade reaction, $lbf/in^2/in$.

For the development of his theory, Westergaard used following assumptions:

- The concrete slab is acting as a homogeneous, isotropic elastic solid in equilibrium.
- The slab cross section is uniform.
- There are no shear or frictional forces.
- There are no in-plane forces.
- The neutral axis is located at the mid-depth of the slab.
- Plain strain assumption is applied.
- Shear deformations are small and can be ignored.
- The slab is considered infinite for the center loading condition and semi-infinite for the edge loading condition.
- The slab is placed on a Winkler foundation in which the subgrade is represented as discrete springs beneath the slab.
- The loads at the interior and the corner of the slab are distributed uniformly over a circular area of contact, whereas the load at the edge of the slab is distributed uniformly over a semicircular area of contact.

There are also several limitations to this theory listed as follows:

- Only deformations and stresses at interior, edge, and corner locations can be calculated.
- Shear and frictional forces on the slab surface may actually be quite considerable.
- The Winkler foundation only extends to the edge of the slab. In reality, support is provided by the surrounding sub-base and subgrade.

- The theory assumes that the slab is fully supported. However, voids or discontinuity exist beneath the slab.
- Load transfer between joints or cracks is not considered in the stress or deflection calculations.

2.2.2 Influence Charts

Based on *Pickett and Ray's Analysis* [2.13] in 1951 influence charts for determining the stress and deflections in concrete pavements are developed. Pickett and Ray used Westergaard's theory and developed theoretical solutions for concrete slabs on an elastic half space and used these solutions in their charts for determining stresses for edge and interior loading conditions. The use of the charts involves the original configuration of contact area which is not the circular area but the original tire imprints. The total number of blocks counted under the contact area related to the estimation of the stress and deflection of the concrete pavement under that wheel load. These charts were used by the Portland Cement Association (PCA) for pavement design in 1966. After *Pickett and Badaruddin* [2.14] a simple influence chart based on solid foundations was developed.

2.2.3 Numerical Methods

Closed-form equations and influence charts assume that the slab and subgrade are in full contact. Due to their simplicity, closed-form equations and influence charts were used to develop simple equations by Westergaard and other researchers at first. However, because of the temperature curling and pumping and moisture warping, the slab and subgrade are usually not in full contact [2.4]. Thus this assumption is unrealistic and does not represent the actual soil behavior. Later, with the development of computer technology, more realistic models could be numerically represented. With the advances in computers, new pavement design methods have been developed for partial contact of the subgrade layer.

Hudson and Matlock [2.15] used a discrete element method to describe the subgrade as a combination of elastic joints, rigid bars, and torsional bars representing subgrade as dense liquid. *Cheung and Zienkiewics* [2.16] developed finite element methods for analyzing pavements on elastic foundations. Finite element method solutions were used to convert the pavement systems into small elements that are connected with structural nodes. The stress and deflections calculated at each nodes resulted is overcoming the previous models limitations. Furthermore, *Huang and Wang* [2.17-18] applied finite element methods on the jointed slabs on liquid foundations. In 1978 *Tabatabaie* [2.19] developed the ILLI-SLAB program. ILLI-SLAB is a finite element program using 2D thin plate elements for the analysis of pavements. *Chou* [2.20] developed finite element programs called WESLIQUID and WESLAYER for the analysis of the liquid and layered foundations, respectively. RISC, KENSLAB and KENLAYER were the other finite elements methods using 2D thin plate elements. Recently *Chen et al. [2.21] and General Accounting Office* in 1997 both have used the 3D finite element modeling for pavement design. Although there are many advantages of using a 3D finite element, due to the computational difficulties and complex modeling problems, they are not adopted for pavement analysis. Commonly used ILLI-SLAB, WELIQUID and WESLAYER, RISC and KENSLABS finite element computer programs are described as follows.

2.2.3.1 ILLI-SLAB Finite Element Model

The most widely used and verified 2D thin plate finite element program, the ILLI-SLAB, was developed at the University of Illinois in the late 1970's for the structural analysis of jointed concrete slabs consisting of one or two layers, with either smooth interface or complete bonding between layers. The model was based on the classical theory medium thick elastic plate on top of a Winkler foundation in its original version. Later the model was revised and improved through several research studies. These studies resulted in the addition of different subgrade models [2.22-23] and in the addition of added capability of linear and non-linear temperature loadings of multi slab layered pavements [2.24]. The program can handle up to 10 slabs in each direction, with joints treated as rectangular elements with zero width. The capabilities of the ILLI-SLAB provide several options for analyzing the following pavement design models:

- Multiple axle loads in any configuration, and axles in any location on the slab

- Jointed plain concrete pavements with longitudinal and transverse cracks with different Load transfer efficiencies (LTE)
- Variable concrete slabs, subgrade supports
- A linear temperature gradient in uniformly thick slabs
- Concrete shoulders with or without tie bars.

2.2.3.2 WESLIQUID and WESLAYER Finite Element Models

In collaboration with *Huang and Chou* [2.20] developed the WESLIQUID and WESLAYER in 1981 at Waterways Experiment Station. The WESLIQUID finite element computer program was developed for the analysis of concrete pavements subjected to the multiple-wheel loads and temperature gradients. WESLAYER, on the other hand, was developed for the computation of state of the stress in a rigid supported on an elastic solid or layered elastic foundation. WESLAYER's method of solution is very similar to the WESLIQUID. WESLIQUID model employs a Winkler foundation, whereas the foundation is considered to be layered in WESLAYER which is more realistic when layers of base and sub-base exist above the subgrade. Multiple slabs and two layer systems with bonded or un-bonded interfaces can be analyzed by WESLIQUID. Slab thicknesses and subgrade moduli may vary from node to node. Curling analysis can be performed under a linear temperature distribution through the thickness of a one or two layer system in both models.

2.2.3.3 RISC Finite Element Model

RISC finite element program was developed in 1983 by Majidzadeh et al. [2.25] as a part of a mechanistic design procedure for rigid pavements. It is based on coupling of a finite element slab on top of a multilayer elastic solid foundation where the slab is represented by thin shell elements. Pavement materials were modeled as linearly elastic, and environmental effects were also considered through the AASHTO regional factor by modifying traffic. RISC is capable of analyzing various rigid pavement sections with various thicknesses and various base, sub-base, and subgrade.

2.2.3.4 KENSLABS Finite Element Model

KENSLABS was developed by *Huang [2.26]* at the University of Kentucky. It can analyze nine slabs with shear and moment transfer across the joints. The program can model slabs on liquid, solid, or layered foundations. It can analyze two layers and slab thickness can vary from node to node or from slab to slab. The unique feature of the program is its ability to perform a damage analysis with up to 24 seasonal periods per year.

2.2.4 Road Tests

The mid 1940s was the start of a new era for pavement design methodologies based on the large scale road tests. The design methods were developed from the observed performance of the pavements under controlled conditions during the road tests. Pavement engineers had the chance for a better understanding of pavement performance under different conditions. All road tests were supervised by the Highway Research Board with the assistance of universities and other trade associations. A few of the more pertinent findings of such test roads which have led or will lead to changes in pavement design include (1) the Maryland road test for rigid pavements, (2) the WASHO road test for flexible pavements, and (3) the AASHO road test for both rigid and flexible pavements. The Maryland road test and AASHO road test were presented in brief, and then the limitations of AASHO test were discussed.

2.2.4.1 Maryland Road Test

The Maryland road test was conducted in 1950 on a 1.1 mile section of US 301 located approximately 9 miles south of La Plata, Maryland. The aim of this road test was to determine the relative effects of four different axle loadings using two vehicle types on a specific concrete pavement design [2.27]. The loads employed were 18,000 pounds and 22,400 pounds on single axles, and 32,000 pounds and 44,000 pounds on tandem axles. These loadings were selected to represent conditions of expected future values on these roads. The major findings indicated that the pumping was the major distress for the pavements on fine-grained soils. The stresses formed on the slab were increased extremely and caused rupture on the slab after pumping occurred.

2.2.4.2 AASHO Road Test

The AASHO road test was the last of the major road tests in the United States, conducted from 1958 to 1960 near Ottawa, Illinois about 80 miles southwest of Chicago [2.28]. The aim of this road test was to identify the relationship between the number of repetition of specified axle loads with different magnitudes and arrangements and pavement thickness. This road test involved both rigid and flexible pavements. Planning the project began about 1950, the site was selected in 1954; construction was carried between 1956 and 1958. Testing began in October 1958 and ended 1960 and data analysis and final reporting were completed in 1962. In all, the test road contained six loops, each with two lanes. Single-axle loadings ranged from 2,000 to 30,000 lb; tandems from 24,000 to 48,000 lb. Field testing and measurement, laboratory work, and analysis of data made use of the most modern equipment and statistical methods. The final reports totaled more than 1600 pages. One of the important findings of the AASHO road test was that the engineers developed the concept of "serviceability ratings" which the smoothness and ride-ability of the various pavement sections were keyed.

2.2.4.2.1 Limitations

The AASHO road test was the most comprehensive of the road tests, yet it was still limited to the influence of only the environment of Central Illinois, the roadbed soil of Central Illinois, and the materials of Central Illinois which were used to construct the pavement sections. One immediate concern was to develop expanded criteria which would allow different conditions and materials to be considered in the design process. Components of the design procedure requiring local verification include:

- Climate
- Soil properties
- Material properties

The basic principles established and validated by the road test still serve as the basis for a large number of performance-based design procedures being used in the United States today. The AASHTO interim guide design for rigid and flexible pavement, Corps of Engineers, Louisiana, Utah, and Kentucky designs are among a large family of pavement design techniques which were primarily developed on the basis of field performance taken from the road test. Their popularity indicates the usefulness of the data collected on the road test.

2.3 Pavement Design Guides

The most widely used procedure for design of concrete pavements is specified in the *Guide for Design of Pavement Structures* published in 1986 and 1993 by the American Association of State Highway and Transportation Officials (AASHTO) [2.29-30]. The 1993 version differs from the 1986 version only in the overlay design chapter. Only a few states use the 1972 AASHO Interim Guide procedure [2.31] or the Portland Cement Association (PCA) procedure [2.32] or their own empirical or mechanistic-empirical procedure, or a design catalog.

2.3.1 AASHTO Design Guides for Pavement Structures

Based on the Results of AASHO road tests an empirical pavement design guide, the 1972 AASHO Interim guide, was published. Basically, the number of axle load applications are used as a function of the slab thickness, axle type (single or tandem) and weight, and terminal serviceability. This original model applies only to the designs, traffic conditions, climate, subgrade, and materials of the AASHO road test. In later versions, it has been extended to make possible the estimation of allowable axle load applications to a given terminal serviceability level for conditions of concrete strength, subgrade *k*-value, and concrete *E* different than those of the AASHO road test. The AASHTO design methodology has also been extended to accommodate the conversion of mixed axle loads to equivalent 80-kN (18-kip) equivalent single axle loads (ESALs) through the use of load equivalency factors. The loss of serviceability that corresponds to a predicted number of axle load applications does not include any contribution of faulting to pavement roughness because the AASHO road test experienced substantial loss of support. The design loss of serviceability is assumed to be entirely due to slab cracking.

2.3.1.1 AASHTO Design Guide – 1986-1993

Due to the limitations of the 1972 interim Design Guide, extensive revisions were made to include more fundamental concepts (some recommended in mechanistic approaches) and extend the applicability of the design procedure. These revisions include:

1. Replacement of soil support value and the modulus of subgrade reaction with the modulus of resilience for both flexible and rigid pavements.
2. The inclusion of design reliability.
3. The use of resilient Modulus testing to select layer coefficients for flexible pavements.
4. Drainage has been included through recognition of the impact of drainage on performance and suitable adjustments to material properties.
5. Improved environmental design has been included for frost heave, swelling soils, and thaw weakening.
6. Load transfer can be designed for in rigid pavements.

7. Life-cycle cost information has been included for use in evaluating alternate designs.

Other items in the design guide which have been added or expanded include rehabilitation, pavement management, load equivalency factors, traffic considerations, and low volume road design.

2.3.1.2 Supplement to AASHTO Design Guide – 1998

The revised AASHTO design model for concrete pavements presented in the 1998 *Supplement to the AASHTO Guide* [2.33] was developed under NCHRP Project 1-30 [2.34] and field-validated by analysis of the GPS-3, GPS-4, and GPS-5 (JPCP, JRCP, and CRCP) sections in the Long-Term Pavement Performance (LTPP) studies [2.35].

The purpose of the NCHRP Project 1-30 study was to evaluate and improve the AASHTO Guide's characterization of subgrade and base support. The original AASHO empirical model was calibrated to the springtime *k*-value measured in plate load tests on the granular base, whereas the 1986 Guide's method for determining the design *k*-value was based on a seasonally adjusted annual average *k*-value. A key recommendation of the 1-30 study was that, subgrade model under rigid pavement design module should be characterized by the seasonally adjusted annual average static elastic values. The 1998 AASHTO Supplement presents guidelines for determination of an appropriate design *k*-value on the basis of plate bearing tests, correlations with soil types and properties, CBR, or deflections measured on in-service pavements. It is recommended in the 1998 AASHTO Supplement that both the beneficial and detrimental effects of a granular or treated base and the computation of slab stress in response to load as well as temperature and moisture gradients should be considered.

2.3.2 Portland Cement Association (PCA) Guidelines

The PCA procedure was developed using the results of finite element analyses of stresses induced in concrete pavements by joint, edge, and corner loading. The PCA procedure, like the 1986-1993 AASHTO procedure, employs the "composite *k*" concept in which the design *k* is a function of the subgrade soil *k*, base thickness, and base type (granular or cement treated). The pavement design procedure has control criteria with respect to two potential failure modes: fatigue and erosion.

The fatigue analysis incorporates the assumption that approximately 6% of all truck loads will pass sufficiently close to the slab edge to produce a significant tensile stress. The fatigue model was changed to eliminate a discontinuity in the high load levels in the current PCA procedure. The erosion analysis quantifies the rate of work with which a slab corner is deflected by a wheel load as a function of the slab thickness, foundation *k*-value, and estimated pressure at the slab-foundation interface. An additional safety factor can be applied to the axle load levels used in the fatigue and erosion analyses to account for the more significant consequences of error in traffic prediction for higher-volume facilities. An adequate thickness is one for which the sum of the contributions of all axle load levels to fatigue and erosion damage is less than 100%.

2.3.3 Mechanistic-Empirical Pavement Design Methods

Mechanistic pavement design procedures are based on mechanics of materials equations that relate an input to pavement response such as stress, strain or deformation (see chapter 1). Laboratory testing is often included to provide relationship between loadings and failure. Empirical design methods (see chapter 1) typically relate observed field performance to design variables, such as a road test. Mechanistic-empirical design approaches combine the theory and physical testing with the observed performance to design the pavement structure.

The basis of a mechanistic-empirical design procedure is to analytical calculation of the stress or strain and transfer these mechanistic stress, strain to the pavement responses using transfer functions to predict distresses resulting from the response. Transfer functions can be developed from laboratory test data or they can be based on observed performance data collected in the field. As more distress survey data becomes available, theoretical models may be more accurately calibrated to represent observed performance models. Calibration with field performance is a necessity for accurate designs as theory alone has not proven sufficient to design pavements realistically.

2.3.4 Design Catalogs

A design catalog does not present a thickness design procedure by itself. It is a format for recommended thicknesses and other design details. A design catalog for both flexible and rigid pavements in the United States was developed under NCHRP Project 1-32 [2.40].

2.3.5 Other Methods

Other concrete pavement design methods are ranging from empirical methods to mechanistic-empirical methods. Most notable among the mechanistic-empirical methods are the zero-maintenance design procedure [2.36-37] and the NCHRP Project 1-26 procedure [2.38-39].

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CHAPTER 3

INPUT PARAMETERS FOR THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

3.1 Introduction

Many design methods do not consider the effect of different climatic locations and material characteristics. This is due to the limited conditions of the AASHO road test in terms of one climate, and one soil condition. In this chapter, the major rigid pavement design input parameters of the mechanical-empirical pavement design guide (MEPDG) are discussed in detail. The rigid pavement design inputs are described under three major categories: (1) Traffic, (2) Climate, and (3) Material inputs. Another aspect of the MEPDG described in this chapter is the hierarchical approach to the design inputs, which is not found in the AASHTO design guides. With this approach, the inputs are separated into three levels as stated in MEPDG [3.1].

3.2 Design Inputs

Design inputs consist of general inputs and three major categories of traffic, climate and material inputs as shown in Figure 3.1 below. Each of the input sections is discussed below.

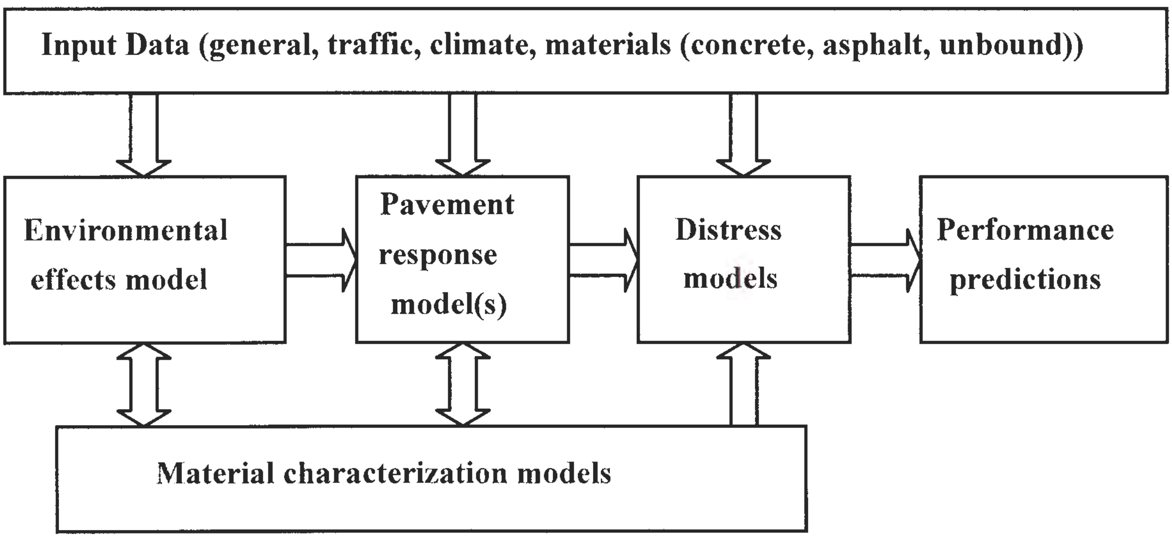

Figure 3.1 Mechanistic-empirical pavement design guide inputs diagram [3.1]

3.2.1 General Inputs

The general inputs section of the MEPDG is categorized into general information, site/project identification information, and the analysis parameters. General information consists of information about the pavement type, design life, and time of construction. In the analysis of parameter tab, limits and reliability values are need to be entered.

3.2.2 Traffic Module

Traffic data is one of the most essential aspects of pavement design. Traffic data required by the MEPDG are in agreement with the *Traffic Monitoring Guide (TMG)*. The traffic loads applied to pavement during its design life and the frequency of vehicle loads is calculated by using the traffic data. The equivalent single axle load (ESAL) used in the different versions of the AASHTO Guide for Pavement Design is not applicable in the MEPDG. MEPDG outputs the accumulated amount of heavy traffic on a monthly basis for the magnitude of truck traffic loadings in the design lane [3.1].

3.2.2.1 Traffic Characterizations Sources

Traffic data is collected by three different methods: (1) weigh-in-motion (WIM), (2) automatic vehicle classification (AVC), and (3) vehicle counts. This data can be augmented by traffic estimates computed using traffic forecasting and trip generation models. Following are the main sources of traffic data that are typically used for the traffic characterization in the MEPDG.

3.2.2.1.1 Weight-In-Motion (WIM) Data

WIM data, providing traffic data over a period of time, includes:

- Vehicle type and the number
- Speed
- Axle weights and gross weight
- Axle spacing

3.2.2.1.2 Automatic Vehicle Classification (AVC) Data

AVC data includes the number and types of vehicles counted over a period of time. AVC data is used to determine the normalized truck class distribution. AVC data can be Level 1 to 3 depending on where the data is collected.

3.2.2.1.3 Vehicle Counts

Vehicle counts are a count of the total number of vehicles categorized by passenger vehicles, buses, and trucks over a period of time. Vehicle counts are used when detailed truck traffic data are unavailable. Thus, it can be either input level 2 or level 3 based on the specific location (site-specific, regional/statewide, or national) where data is collected.

3.2.2.1.4 Traffic Forecasting and Trip Generation Models

Traffic forecasting and trip generation models can be used for estimation of Level 1 or Level 2 type of data used in the MEPDG depending on their calibration of site-specific or regional/statewide data.

3.2.2.2 Traffic Characterization Inputs

Four basic types of traffic data are required for pavement structural design: (1) Traffic volume, (2) Traffic volume adjustment factors, (3) Axle load distribution factors, and (4) General traffic inputs (see Figure 3.2).

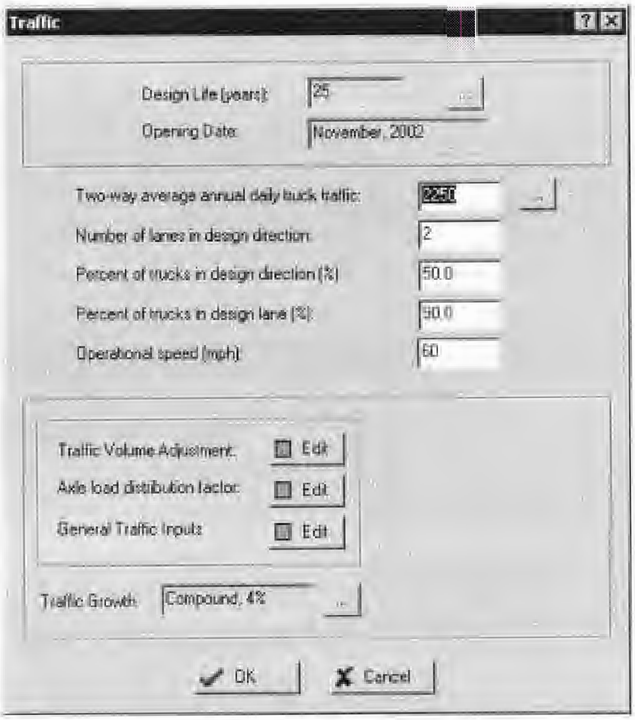

Figure 3.2 Screenshot of MEPDG software for traffic characterization inputs

3.2.2.2.1 Traffic volume

The base year for the traffic inputs is defined as the first calendar year that the roadway segment under design is opened to traffic. The following base year information is required:

- Two-way annual average daily truck traffic (AADTT).
- Number of lanes in the design direction.
- Percent trucks in design direction.
- Percent trucks in design lane.
- Vehicle (truck) operational speed.

3.2.2.2.1.1 Two-Way Annual Average Daily Truck Traffic (AADTT)

The total number of heavy vehicles of classes 4 to 13 in the traffic stream passing a point or segment of a road in both directions during a 24-hour period is called two-way annual average daily truck traffic (AADTT). It is commonly obtained simply by dividing the total number of truck traffic of the given time period by the number of days in that time period. Base year AADTT is defined as Level 1, 2 or 3. The input level is based on the level of the sources (WIM, AVC, Vehicle Counts, or Traffic forecasting and trip generation models). Local experience is also considered as Level 3 data.

3.2.2.2.1.2 Number of Lanes in the Design Direction

The number of lanes in the design direction is determined from design specifications and represents the total number of lanes in one direction.

3.2.2.2.1.3 Percent Trucks in Design Direction

This design input defines the percentage of trucks in the design direction. The directional distribution factor (DDF) can be used to calculate the difference in the different directions. It is usually assumed to be 50% when traffic is given in two directions; however, this is not always the case. The MEPDG software provides a default value (Level 3) of 55% for interstate-type facilities computed using traffic data from the LTPP database [3.2-3]. The levels of input for percent trucks in design direction are defined Level 1 through 3 depending on the level of DDF determined from traffic source levels.

3.2.2.2.1.4 Percent Trucks in Design Lane

Percent trucks in the design lane, or truck lane distribution factor (LDF), accounts for the distribution of truck traffic between the lanes in one direction. For two-lane, two-way highways (one lane in one direction), this factor is 1.0 because all truck traffic in any one direction must use the same lane. For multiple lanes in one direction, it depends on the

AADTT and other geometric and site-specific conditions. The level of input for LDF is based on the source data. [3.1]

The default (Level 3) values recommended for use based on the LDF for the most common type of truck (vehicle class 9 trucks) is as follows:

- Single-lane roadways in one direction, LDF = 1.00.
- Two-lane roadways in one direction, LDF = 0.90.
- Three-lane roadways in one direction, LDF = 0.60.
- Four-lane roadways in one direction, LDF =0.45.

3.2.2.2.1.5 Vehicle Operational Speed

The average vehicle speed in the MEPDG is given as 60 mph, but this value can be modified to reflect local conditions. A description of a detailed methodology used for determining operational speeds can be found in the Transportation Research Board *Highway Capacity Manual* or AASHTO's *A Policy on Geometric Design of Highways and Streets* (often called the "Green Book") [3.4-5].

3.2.2.2.2 Traffic Volume Adjustment Factors

The following truck-traffic volume adjustment factors are required for traffic characterization:

- Monthly adjustment.
- Vehicle class distribution.
- Hourly truck distribution.
- Traffic growth factors.

3.2.2.2.2.1 Monthly Adjustment Factors

Truck traffic monthly adjustment factors are the percentage of the annual truck traffic for a given truck class in a specific month. Monthly adjustment factors (MAF) can be calculated regardless of the source of the data (WIM, AVC, vehicle count, and so on), each for different types of highways as follows [3.1]:

- For the given traffic data (24-hour of continuous data collection), determine the total number of trucks (in a given class) for each 24-hour period. If data were not collected for the entire 24-hour period, the measured daily truck traffic should be adjusted to be representative of a 24-hour period.
- Using representative daily data collected for the different months within a year, determine the average daily truck traffic for each month in the year.
- Sum up the average daily truck traffic for each month for the entire year.

- Calculate the monthly adjustment factors by dividing the average daily truck traffic for each month by summing the average daily truck traffic for each month for the entire year and multiplying it by 12 as given below:

$$MAF_i = \frac{AADTT_i}{\sum_{i=1}^{12} AADTT_i} * 12$$

Where,

MAF_i = monthly adjustment factor for month i

$AADTT_i$ = AADTT for month i

The sum of the MAF of all months must equal 12. Pavement designs can be sensitive to MAF. If no information is available, it is recommended that designers assume an even or equal distribution (i.e., 1.0 for all months for all vehicle classes).

3.2.2.2.2.2 Vehicle Class Distribution

The data obtained from such AVC, WIM, and vehicle counts are used to obtain vehicle classification. Figure 3.3 shows the standard vehicle classes that have been used for FHWA and LTPP [3.2-3].

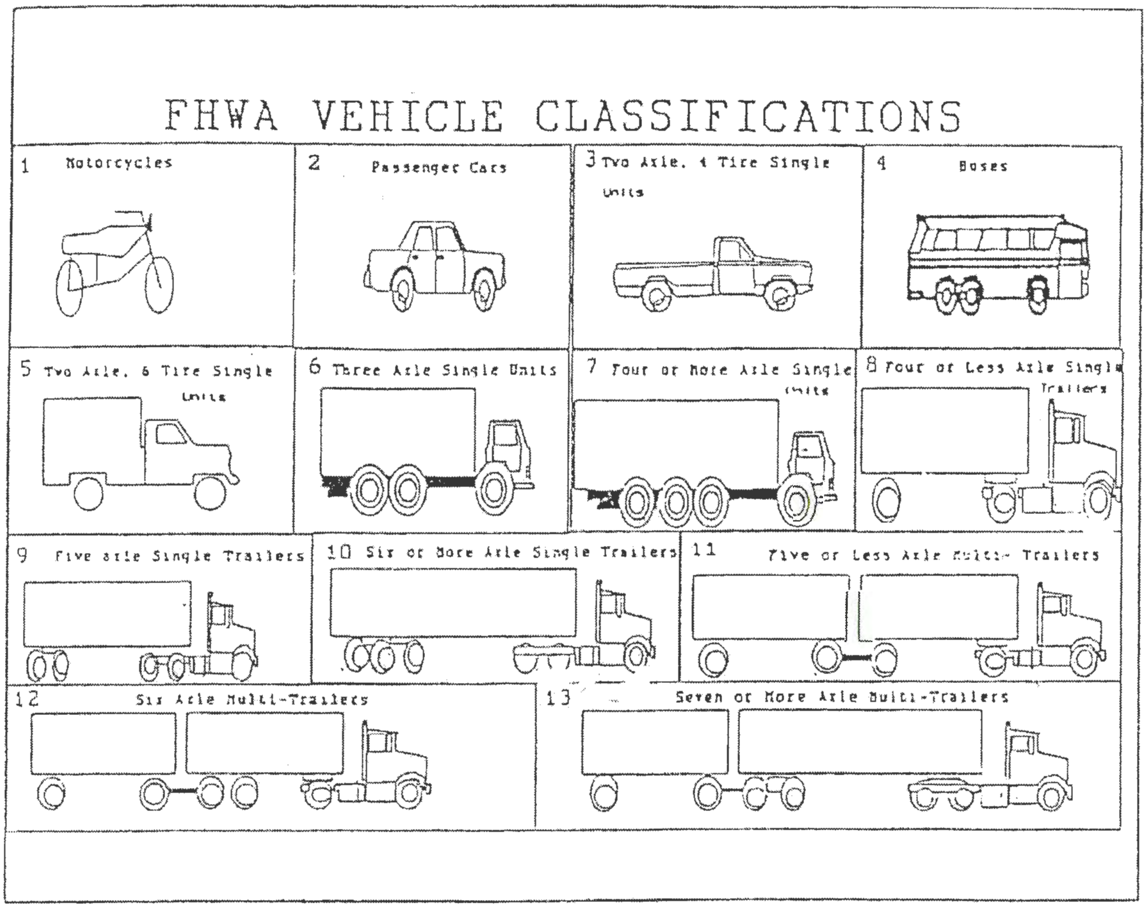

Figure 3.3 Illustrations and definitions of the vehicle classes used for collecting traffic data that are needed for design purposes [3.1].

3.2.2.2.2.3 Truck Hourly Distribution Factors

The hourly distribution factors (HDF) represent the percentage of the traffic within each hour of the day. The sum of the percent of daily truck traffic per time increment must add up to 100 percent.

3.2.2.2.2.4 Traffic Growth Factors

Traffic growth factors represent the future estimates of the traffic data. The MEPDG software allows users to use three different traffic growth functions to compute the growth or decay in truck traffic over time (forecasting truck traffic). The three functions provided to estimate future truck traffic volumes are presented as follows:

$$AADTT_X = 1.0 * AADTT_{BY} \text{ (No growth)}$$

$$AADTT_X = GR * AGE + AADTT_{BY} \text{ (Linear growth)}$$

$$AADTT_X = ADTT_{BY} * (GR)^{AGE} \text{ (Compound growth)}$$

Where;

$AADTT_X$ = Annual average daily truck traffic at age X

GR = Traffic growth rate

$AADTT_{BY}$ = Base year annual average daily truck traffic

3.2.2.2.3 Axle Load Distribution Factors

The axle load distribution factors basically correspond to the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (classes 4 through 13). The load intervals for each axle type are provided below [3.1]:

- Single axles – 3,000 lb to 40,000 lb at 1,000-lb intervals.
- Tandem axles – 6,000 lb to 80,000 lb at 2,000-lb intervals.
- Tridem and quad axles – 12,000 lb to 102,000 lb at 3,000-lb intervals.

3.2.2.2.4 General Traffic Inputs

General traffic inputs can be summarized as follows:

- Lateral traffic wander
- Number of axle types per truck class
- Axle configuration
- Wheel base

3.2.2.2.4.1 Lateral Traffic Wander

Traffic wander effect is defined with 3 inputs: (1) Mean wheel location, (2) Traffic wander standard deviation, and (3) Design lane width. The *Mean wheel location* is the distance from the outer edge of the wheel to the pavement marking. 18 inch of recommended default value is provided with the MEPDG software. *Traffic wander standard deviation* is the statistic describing how tightly the lateral traffic wander is clustered around the mean wheel location. A default (Level 3) mean truck traffic wander standard deviation of 10 inches is provided in the MEPDG software. This is recommended if more accurate information is not available. *Design lane width* is the parameter that refers to the actual traffic lane width, as defined by the distance between the lane markings on either side of the design lane. It is a design factor and may or may not equal the slab width. The default value for standard-width lanes is 12 ft. [3.1]

3.2.2.2.4.2 Number of Axle Types per Truck Class

This input represents the average number of axles for each truck class (class 4 -13) for each axle type (single, tandem, tridem, and quad). The inputs at different levels are based on the traffic source data.

3.2.2.2.4.3 Axle Configuration

A series of data elements are needed to describe the configurations of the typical tire and axle loads that would be applied to the roadway because computed pavement responses are generally sensitive to both wheel locations and the interaction between the various wheels on a given axle. These data elements can be obtained directly from manufacturers databases or measured directly in the field. Typical values are provided for each of the following elements; however, site-specific values may be used, if available.

- Average axle-width – the distance between two outside edges of an axle. For typical trucks, 8.5 ft may be assumed for axle width.
- Dual tire spacing – the distance between centers of a dual tire. Typical dual tire spacing for trucks is 12 in.
- Axle spacing – the distance between the two consecutive axles of a tandem, tridem, or quad. The average axle spacing is 51.6 inches for tandem and 49.2 inches for tridem and quad axles.

3.2.2.2.4.4 Wheelbase

Vehicles wheelbase can be obtained directly from manufacturer's database or measured in the field. Typical values are provided for the average axle spacing and percent of trucks are provided as follows [3.1]:

- Average axle spacing (ft) – short, medium, or long. The recommended values are 12, 15, and 18 ft for short, medium, and long axle spacing, respectively.
- Percent of trucks in class 8 - 13 with the short, medium, and long axle spacing – use even distribution (e.g., 33, 33, and 34% for short, medium, and long axles, respectively), unless more accurate information is available.

3.2.3 Climate Module

The environmental effects on pavements and pavements' reaction to the environmental conditions have an important effect on the design of rigid pavements. The required parameters can be defined as internal and external inputs. The external inputs are precipitation, temperature, freeze-thaw cycles, and depth of water table. The pavement reactions such as the susceptibility of the pavement materials to moisture and freeze-thaw damage, and drainability and infiltration properties of pavement layers are called internal inputs. These required input parameters are created through a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). The necessary climate inputs are the climatic locations. There are already large numbers of defined locations in the MEPDG but also using latitude and longitude, the climatic data can be generated by extrapolating nearby weather stations (see Figure 3.4). MEPDG software provided 15 climatic weather stations for Iowa including Ames, Des Moines, Iowa City. The program reads hourly climatic information during the analysis stage. The climate file contains the sunrise time, sunset time and radiation for each day of the design life period. In addition, for each 24-hour period in each day of the design life, the temperature, rainfall, air speed, sunshine, and depth of ground water table are also listed in the climate file. With this information, the EICM computes and predicts the following information for pavement layers: temperature, resilient modulus adjustment factors, pore water pressure, water content, frost and thaw depths, frost heave, and drainage performance.

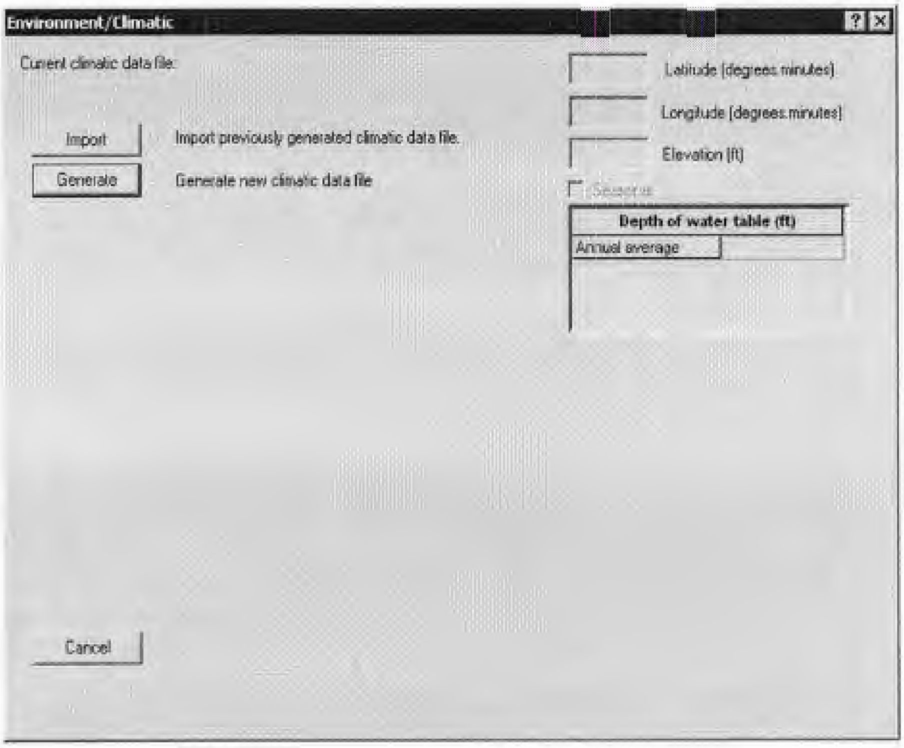

Figure 3.4 Screenshot of climatic module of the MEPDG software

3.2.4 Materials Module

The major categorical system developed for the M-E Pavement Design Guide is presented in Table 3.1. Six major material groups have been developed: asphalt materials, PCC materials, cementitiously or chemically stabilized materials, non-stabilized granular materials, subgrade soils, and bedrock.

Table 3.1 Material types used in the MEPDG [3.1]

Asphalt Materials * Hot Mix AC—Dense Graded Central Plant Produced In-Place Recycled * Hot Mix AC—Open Graded Asphalt * Hot Mix AC—Sand Asphalt Mixtures * Cold Mix AC Central Plant Processed In-Place Recycled	Non-Stabilized Granular Base/Subbase * Granular Base/Subbase * Sandy Subbase * Cold Recycled Asphalt (used as aggregate) RAP (includes millings) Pulverized In-Place * Cold Recycled Asphalt Pavement (AC plus aggregate base/subbase)
PCC Materials * Intact Slabs * Fractured Slabs Crack/Seat Break/Seat Rubblized Cementitiously Stabilized Materials * Cement Stabilized Materials * Soil Cement * Lime Cement Fly Ash * Lime Fly Ash * Lime Stabilized/Modified Soils * Open graded Cement Stabilized Materials	Subgrade Soils * Gravelly Soils (A-1;A-2) * Sandy Soils Loose Sands (A-3) Dense Sands (A-3) Silty Sands (A-2-4;A-2-5) Clayey Sands (A-2-6; A-2-7) * Silty Soils (A-4;A-5) * Clayey Soils Low Plasticity Clays (A-6) Dry-Hard Moist Stiff Wet/Sat-Soft High Plasticity Clays (A-7) Dry-Hard Moist Stiff Wet/Sat-Soft Bedrock * Solid, Massive and Continuous * Highly Fractured, Weathered

PCC and unbound granular and subgrade material inputs used in the MEPDG are described briefly below.

3.2.4.1 Portland Cement Concrete

PCC inputs of MEPDG are gathered under 4 headings: strength inputs, general inputs, mix design inputs, and thermal inputs. These parameters will be explained in detail.

3.2.4.1.1 Strength Parameters for PCC Materials

Modulus of elasticity and flexural strength of PCC materials are the main strength parameters used in the MEPDG software. The detailed information for the calculation of these inputs is given next.

3.2.4.1.1.1 Modulus of Elasticity

The ratio of stress to strain in the elastic range of a stress-strain curve for a given concrete mixture defines its modulus of elasticity [3.6]. The PCC modulus of elasticity is influenced significantly by mix design parameters and mode of testing. The mixture parameters that most strongly influence elastic modulus include ratio of water to cementitious materials, and relative proportions of paste and aggregate. For each hierarchical level of inputs the procedure of estimating PCC elasticity modulus (E_c) differs as below.

PCC elastic modulus values estimated from laboratory testing for input level 1. The modulus values at 7, 14, 28, and 90 days are required. In addition, the estimated ratio of 20-year to 28-day E_c is also a required input (a maximum value of 1.20 is recommended for this parameter). The recommended test procedure for obtaining E_c is ASTM C 469, *Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.* The required input data at Level 1 for this parameter are summarized in Table 3.2.

Table 3.2 Required input data for modulus of elasticity at level 1 [3.1]

Input parameter	Required test data				Ratio of 20-yr/28-day modulus	Recommended test procedure
	7-day	14-day	28-day	90-day		
E_c	✓	✓	✓	✓	✓	ASTM C469

For input Level 2, E_c can be estimated from compressive strength (f'_c) testing through the use of standard correlations. Static elastic modulus can be estimated from the compressive strength of the PCC using the American Concrete Institute (ACI) equation:

$$E_c = 33\,\rho^{3/2}\,(f'_c)^{1/2}$$

Where,

Ec = PCC elastic modulus, psi.

ρ = unit weight of concrete, lb/ft^3.

f'_c = compressive strength of PCC, psi.

Input compressive strength results at 7, 14, 28, and 90 days and the estimated ratio of 20-year to 28-day compressive strength are required. Testing should be performed in accordance with AASHTO T 22, *compressive strength of cylindrical concrete specimens*. Table 3.3 summarizes the recommended procedures and required input data at Level 2.

Table 3.3 Required input data for modulus of elasticity at level 2 [3.1]

Input parameter	Required test data				Ratio of 20-yr/28-day strength	Recommended test procedure
	7-day	14-day	28-day	90-day		
Compressive strength	✓	✓	✓	✓	✓	AASHTO T 22

Estimating PCC Elastic Modulus at Input Level based on a single point (28-day) estimate of the concrete strength (either modulus of rupture (MR) or f'_c) using strength gain equations:

$$STRRATIO = 1.0 + 0.12*\log_{10}(AGE/0.0767) - 0.01566*[\log_{10}(AGE/0.0767)]^2$$

$$MR = 9.5\,(f_c)^{0.5} \text{ (MR and } f'_c \text{ in psi)}$$

Where,

STRRATIO = strength ratio of MR at a given age to MR at 28 days.

AGE = specimen age in years.

Additionally, if the 28-day Ec is known for the project mixtures, it can also be input to better define the strength-modulus correlation. Table 3.4 summarizes the recommended input data at Level 3.

Table 3.4 Required input data for modulus of elasticity at level 3 [3.1]

Input parameter	28-day value	Recommended test procedure
Flexural Strength	✓	AASHTO T97 or from records
Compressive strength	✓	AASHTO T22 or from records
Elastic modulus	Optional – to be entered with either the flexural or compressive strength inputs	ASTM C469 or from records

3.2.4.1.1.2 Flexural Strength of PCC Materials

The flexural strength, often termed modulus of rupture (MR), can be defined as the maximum tensile stress at rupture at the bottom of a simply supported concrete beam during a flexural test with third point loading [3.1]. Like all measures of PCC strength, the modulus of rupture is strongly influenced by mix design parameters. Table 3.5 summarizes the required input data for different input levels for modulus of rupture estimation.

Table 3.5 Modulus of rupture estimation for different level of inputs [3.1]

Input Level	Description
1	• PCC MR will be determined directly by laboratory testing using the AASHTO T 97 protocol at various ages (7, 14, 28, 90-days). • Estimate the 20-year to 28-day (long-term) MR ratio. • Develop strength gain curve using the test data and long-term strength ratio to predict MR at any time over the design life.
2	• PCC MR will be determined indirectly from compressive strength testing at various ages (7, 14, 28, and 90 days). The recommended test to determine f'_c is AASHTO T 22. • Estimate the 20-year to 28-day compressive strength ratio. • Develop compressive strength gain curve using the test data and long-term strength ratio to predict f'_c at any time over the design life. • Estimate MR from f'_c at any given time using the following relationship: $MR = 9.5 * (f'_c)^{1/2}$ psi
3	• PCC flexural strength gain over time will be determined from 28-day estimates of MR or f'_c. • If MR is estimated, use the equation below to determine the strength ratios over the pavement design life. The actual strength values can be determined by multiplying the strength ratio with the 28-day MR estimate. $STRRATIO = 1.0 + 0.12\log_{10}(AGE/0.0767) - 0.01566[\log_{10}(AGE/0.0767)]^2$ • If f'_c is estimated, convert f'_c to MR using equation 2.2.28 and then use the equation above to estimate flexural strength at any given pavement age of interest.

3.2.4.1.2 General Input Parameters

General input parameters are poisson's ratio, unit weight, and PCC layer thickness. The poisson's ratio and unit weight are discussed below. The PCC layer thickness is the user input that can be modified to obtain predefined performance criteria.

3.2.4.1.2.1 Poisson's Ratio of PCC Materials

Poisson's ratio (μ) can be determined either Level 1 or Level 3. At input Level 1, poisson's ratio is determined simultaneously with the determination of the elastic modulus, in accordance with ASTM C 469. Typical values shown in Table 3.6 can be used for Level 3. Poisson's ratio for PCC paving applications ranges between 0.15 and 0.18.

Table 3.6 Typical poisson's ratio values for PCC materials. [3.1]

PCC materials	Level 3 μ_{range}	Level 3 $\mu_{typical}$
PCC Slabs	0.15 – 0.25	0.20
Fractured Slab		
Crack/Seat	0.15 – 0.25	0.20
Break/Seat	0.15 – 0.25	0.20
Rubbilized	0.25 – 0.40	0.30

3.2.4.1.2.2 Unit Weight of PCC Materials

Table 3.7 presents the recommended approaches to determine the unit weight of PCC materials for different levels of input.

Table 3.7 Unit weight estimation of PCC materials [3.1]

Material group category	Input Level	Description
PCC	1	• Estimate value from testing performed in accordance with AASHTO T 121 – Mass per Cubic Meter (Cubic Foot), Yield, and Air Content (Gravimetric) of Concrete
	2	• Not applicable.
	3	• User selects design values based upon agency historical data or from typical values shown below: Typical range for normal weight concrete: 140 to 160 lb/ft^3

3.2.4.1.3 PCC Mix Design Inputs

Mix design inputs are summarized as follows:

- Cement type
- Cementitious material content
- Water/cement ratio
- Aggregate type
- PCC zero-set temperature
- Shrinkage
    - Ultimate shrinkage strain, micro-strain units.
    - Time required to develop 50 percent of the ultimate shrinkage strain
    - Anticipated amount of reversible shrinkage
    - Curing method

Shrinkage can cause significant curling and warping in PCC slabs resulting in pavement cracking.

3.2.4.1.4 PCC Thermal Design Inputs

PCC thermal conductivity, heat capacity, and the coefficient of thermal expansion are the required thermal properties of the PCC layer. The level 1 and 2 values for PCC thermal conductivity and heat capacity is estimated using laboratory testing in accordance with ASTM E 1952 and ASTM D 2766 respectively. For level 3 the recommended values for former ranges from 1.0 to 1.5 Btu/ (ft) (hr) (°F), and latter ranges from 0.2 to 0.28 Btu/ (lb) (°F). The PCC coefficient of thermal expansion is discussed in detail below.

3.2.4.1.4.1 PCC Coefficient of Thermal Expansion

The coefficient of thermal expansion (α_{PCC}) is defined as the change in unit length per degree of temperature change. When the α_{PCC} is known, the unrestrained change in length produced by a given change in temperature can be calculated as [3.1]:

$$\Delta L = \alpha_{PCC} \, \Delta T \, L$$

Where,

ΔL = change in unit length of PCC due to a temperature change of ΔT.

α_{PCC} = coefficient of linear expansion of PCC, strain per °F.

ΔT = temperature change (T_2 - T_1), °F.

L = length of specimen (i.e., joint spacing)

Typical ranges of α is given in Table 3.8.

Table 3.8 Typical α ranges for common PCC components.

Material type	Coefficient of thermal expansion 10^{-6}/°F	Material Type	Coefficient of thermal expansion 10^{-6}/°F
Aggregate		**Cement Paste (saturated)**	
Granite	4-5	*w/c = 0.4*	10-11
Basalt	3.3-4.4	*w/c = 0.5*	10-11
Limestone	3.3	*w/c = 0.6*	10-11
Dolomite	4-5.5	**Concrete**	4.1-7.3
Sandstone	6.1-6.7	**Steel**	6.1-6.7
Quartzite	6.1-7.2		
Marble	2.2-4		

3.2.4.2 Unbound Granular and Subgrade Materials

Unbound granular materials and subgrade materials are chosen according to the unified soil classification (USC) and AASHTO classification of soils in the MEPDG. The AASHTO soil classification is explained in the specifications as the test AASHTO M 145 "The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes." AASHTO soil classification uses the particle-size distributions and consistency limits (Atterberg limits) to classify the soils. AASHTO soil classification is based on the portion of unbound granular and subgrade materials that is smaller than 3-in diameter. The AASHTO classification system identifies two material types:

- Granular materials (i.e., materials having 35% or less, by weight, particles smaller than 0.0029-in in diameter).
- Silt-clay materials (i.e., materials having more than 35%, by weight, particles smaller than 0.0029-in in diameter).

These two divisions are further subdivided into 7 main group classifications (i.e., A-1 though A-7). The group and subgroup classifications are based on estimated or measured grain-size distribution and on liquid limit and plasticity index values.

The USC system is explained in the test standard ASTM D2487, "Standard Method for Classification of Soils for Engineering Purposes." The USC system identifies three major soil divisions:

- Coarse-grained soils (i.e., materials having less than 50%, by weight, particles smaller than 0.0029-in in diameter).
- Fine-grained soils (i.e., materials having 50% or more, by weight, particles smaller than 0.0029-in in diameter).
- Highly organic soils (materials that demonstrate certain organic characteristics).

These divisions are further subdivided into basic soil groups. The major soil divisions and basic soil groups are determined on the basis of estimated or measured values for grain-size distribution and Atterberg limits. ASTM D 2487 shows the criteria chart used for classifying soil in the USC system and the basic soil groups of the system. For this design procedure, unbound granular materials are defined using the AASHTO classification system and are the materials that fall within the specifications for soil groups A-1 to A-3. Subgrade materials are defined using both the AASHTO and USC and cover the entire range of soil classifications available under both systems.

Resilient modulus, M_R, is required for the pavement response model used in the MEPDG as well as poisson's ratio, μ. Those materials are used for the computation of the stress dependent stiffness of unbound granular materials, subgrade materials, and bedrock materials under moving loads. Resilient modulus is defined as the ratio of the repeated deviator axial stress to the recoverable axial strain. They are used to characterize layer behavior when subjected to stresses. Unbound materials display stress-dependent properties (i.e., granular materials generally are "stress hardening" and show an increase in modulus with an increase in stress while fine-grained soils generally are "stress softening" and display a modulus decrease with increased stress).

3.2.4.2.1 Non Linear Material Characterization Models

In pavement design the repeated moving traffic loads are one of the most important factors to be considered. Under repeated loading, most of the deformations are recoverable and thus considered elastic. The resilient modulus (M_r) is then defined as the elastic stiffness of the pavement materials for analysis of repeated traffic loads. In the MEPDG following nonlinear model is used to characterize the resilient modulus of unbound bases, sub-bases, and sub-grades.

$$M_r = k_1 p_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}+1\right)^{k_3}$$

Where;

M_r = resilient modulus

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$

σ_1 = major principal stress

σ_2 = intermediate principal stress = σ_3 for M_R Test on cylindrical specimen

σ_3 = minor principal stress/confining pressure

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}$

P_a = normalizing stress (atmospheric pressure)

k_1, k_2, k_3 = regression constants

The above model is used to fit the laboratory generated M_r test data. This is used in the level 1 input. For level 2 input, general correlations can be used to estimate the M_R value. General correlations are given in the Table 3.9.

Table 3.9 General correlations to find M_R [3.1]

Strength Index Property	Model	Comments	Test standard
CBR	$M_r = 2555(CBR)^{0.64}$	CBR = California bearing Ratio, percent	AASHTO T193—The california bearing ratio
R-value	$M_r = 1155 + 555R$	R = R-value	AASHTO T190—Resistance r-value and expansion pressure of compacted soils
AASHTO layer coefficient	$M_r = 30000\left(\frac{a_i}{0.14}\right)$	a_i = AASHTO layer coefficient	AASHTO guide for the design of pavement structures (1993)
PI and gradation*	$CBR = \frac{75}{1+0.728(wPI)}$	wPI = P200*PI P200= percent passing No. 200 sieve size PI = plasticity index, percent	AASHTO T27—Sieve analysis of coarse and fine aggregates AASHTO T90—Determining the plastic limit and plasticity index of soils
DCP*	$CBR = \frac{292}{DCP^{1.12}}$	CBR = California bearing ratio, percent DCP =DCP index, in/blow	ASTM D6951—Standard test method for use of the dynamic cone penetrometer in shallow pavement applications

*Estimates of CBR are used to estimate M_r.

The Table 3.10 summarizes the recommended values for each soil class.

Table 3.10 Typical modulus values for different soils

Material Classification	M_r Range (psi)	Typical M_r* (psi)
A-1-a	38,500 – 42,000	40,000
A-1-b	35,500 – 40,000	38,000
A-2-4	28,000 – 37,500	32,000
A-2-5	24,000 – 33,000	28,000
A-2-6	21,500 – 31,000	26,000
A-2-7	21,500 – 28,000	24,000
A-3	24,500 – 35,500	29,000
A-4	21,500 – 29,000	24,000
A-5	17,000 – 25,500	20,000
A-6	13,500 – 24,000	17,000
A-7-5	8,000 – 17,500	12,000
A-7-6	5,000 – 13,500	8,000
CH	5,000 – 13,500	8,000
MH	8,000 – 17,500	11,500
CL	13,500 – 24,000	17,000
ML	17,000 – 25,500	20,000
SW	28,000 – 37,500	32,000
SP	24,000 – 33,000	28,000
SW-SC	21,500 – 31,000	25,500
SW-SM	24,000 – 33,000	28,000
SP-SC	21,500 – 31,000	25,500
SP-SM	24,000 – 33,000	28,000
SC	21,500 – 28,000	24,000
SM	28,000 – 37,500	32,000
GW	39,500 – 42,000	41,000
GP	35,500 – 40,000	38,000
GW-GC	28,000 – 40,000	34,500
GW-GM	35,500 – 40,500	38,500
GP-GC	28,000 – 39,000	34,000
GP-GM	31,000 – 40,000	36,000
GC	24,000 – 37,500	31,000
GM	33,000 – 42,000	38,500

3.3 References

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CHAPTER 4

SENSITIVITY ANALYSIS OF RIGID PAVEMENTS MODULE DESIGN INPUT PARAMETERS

4.1 Introduction

The main focus of this chapter was to identify the sensitivity of input parameters needed for designing jointed plain concrete pavements used in the mechanistic-empirical pavement design guide (MEPDG). To study the sensitivity of the large number of input parameters on the predicted pavement distresses, two rigid pavement sections were selected from the Iowa Department of Transportation (Iowa DOT) Pavement Management Information System (PMIS). A history of pavement deflection testing, material testing, traffic, and other related data were also available in the LTPP database. Several hundred sensitivity runs were conducted using the MEPDG software to study the selected rigid pavement sites extensively. For unknown input parameters needed to run the MEPDG software, the nationally calibrated default values were used. Sensitivity analyses were conducted on a standard pavement section formed from two JPCP sites to study the effects on pavement performance in terms of faulting, transverse cracking, and smoothness. Based on the sensitivity analysis of the rigid pavement (Portland cement concrete) input parameters, a sensitivity chart were determined and presented from the most sensitive to insensitive to help the pavement design engineers identify the level of importance of each input parameter. A comparison on predicted pavement smoothens for two Iowa sites using the MEPDG software and the measured pavement distresses values from DOT are presented.

4.2 Data Collection

The very first part of this project was the extensive data collection. From the Iowa Department of Transportation (Iowa DOT) Pavement Management Information System (PMIS) two rigid pavement sections were selected which were also a part of the Long Term Pavement Performance (LTPP) program. A history of pavement deflection testing, material testing, traffic, and other related data were available in the LTPP database. These two sections were named as PCC-1 and PCC-2 (see Table 4.1 and Figure 4.1). Detailed information for these two sites is given in the following headings:

Table 4.1 General information on two selected rigid pavement sites

Section	County	Route	Direction	Beginning Mile Post	Ending Mile Post	Design Year	Project No
PCC-1	Johnson (52)	US218	1	86.03	90.08	1983	F-518-4 (21)-20-52
				90.08	96.8	1983	F-518-4 (12)-20-52
			2	86.03	90.08	1983	F-518-4 (21)-20-52
				90.08	96.8	1983	F-518-4 (12)-20-52
PCC-2	Hamilton (40)	US20	1	149.5	153.47	1968	F-520-4 (7)-20-40
			2	149.5	153.47	1968	F-520-4 (7)-20-40

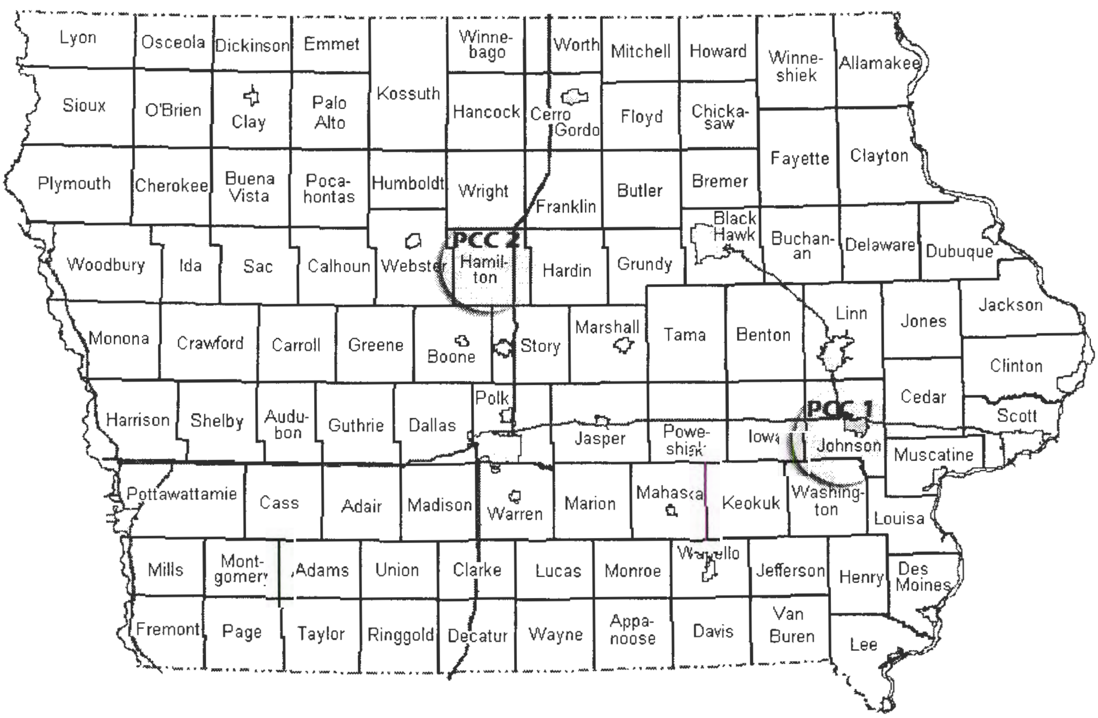

Figure 4.1 Locations of two selected rigid pavement sites in Iowa

4.2.1 PCC-1

PCC-1, located on US-218 near Johnson County, Iowa, was constructed in 1983. The test section was in the northbound direction, and designated between 86.03 and 90.08 miles of US-218. A complete listing of information obtained is summarized in the Tables 4.2, 4.3, 4.4 and 4.5; and Figure 4.2 shows the summary of general information from LTPP data.

4.2.1.1 Traffic

The traffic records provided by the Iowa Department of Transportation indicated that, in 1983, the pavement carried a two-way average daily traffic (ADT) of 2,500 vehicles per day, including heavy trucks. In 2002, it was estimated as 3,590 vehicles per day, including 540 vehicles of truck traffic.

4.2.1.2 Climate

This section of US-218 is located in the wet-freeze environmental region. This area has a freezing index of 466.88, and receives 930.58 mm of rainfall annually. The latitude and longitudes are given as 41.57 and 91.55 degrees respectively.

4.2.1.3 Structure

The pavement is a 9.6-inch JPCP with 15 ft joints and Class II type aggregates. The slab rests on 4 inch (it is mentioned as 4.8 in Treated base in LTPP database) Class A sub-base course. The subgrade of the site is AASHTO A-7-6 material and it is noted that there exists silty clay of Loess material with some glacial till treatments. Modulus of subgrade (k) of this section is taken as 100 pcf in the project files, and the modulus of rupture value from 3rd point loading is noted as 535 psi.

Table 4.2 PCC-1: Location information

County Name:	Johnson County (52)
LTPP Section ID Number:	19-3033-1
LTPP SHRP Region:	North Central
Functional Class:	2
Route Number:	218
Elevation (ft):	641
Latitude (deg.):	41.57
Longitude (deg.)	91.55
Milepost:	86.03 – 96.8 (86.03 – 90.08 – 96.8)

Table 4.3 PCC-1: Pavement information

Construction Date:	8/1/1983
Surface Layer:	9.6 inch PCC (9 ½ project file, Class "C" Pave. with CD joints using Class II aggregate.)
Base Layer:	4.8 inch TB (4 in. Class A sub-base, project file)
Subgrade:	SS layer type. (Silty Clay Loess & Alluvium A-7-6 (12-17) with some glacial till treatments A-6 (12) to A-7-6 (15)
Subgrade k (pcf):	100
Modulus of Rupture from 3rd point loading (psi):	535

Table 4.4 PCC-1: Climate information

Climatic Region:	Wet Freeze
Freezing Index (C-Days):	466.88
Precipitation (mm):	930.58
Days Above 32 C:	24.53
Years of Climatic Data:	17

Table 4.5 PCC-1: Traffic information

Project No:	F-518-4 (21) –20—52
Direction of Traffic:	North Bound
Used Design Method:	DOT spreadsheets using PCA

Table 4.5 Continued

Design Life:	20 years
Designed year:	1982
Designed year Traffic (vpd):	2500
Design Life Traffic (vpd):	3590 (@ 2002)
Design Life Truck Traffic (vpd):	540 (@ 2002)
Design Life Other Traffic (vpd):	3050 (@ 2002)
Traffic Vehicle Distribution and ESALs	-

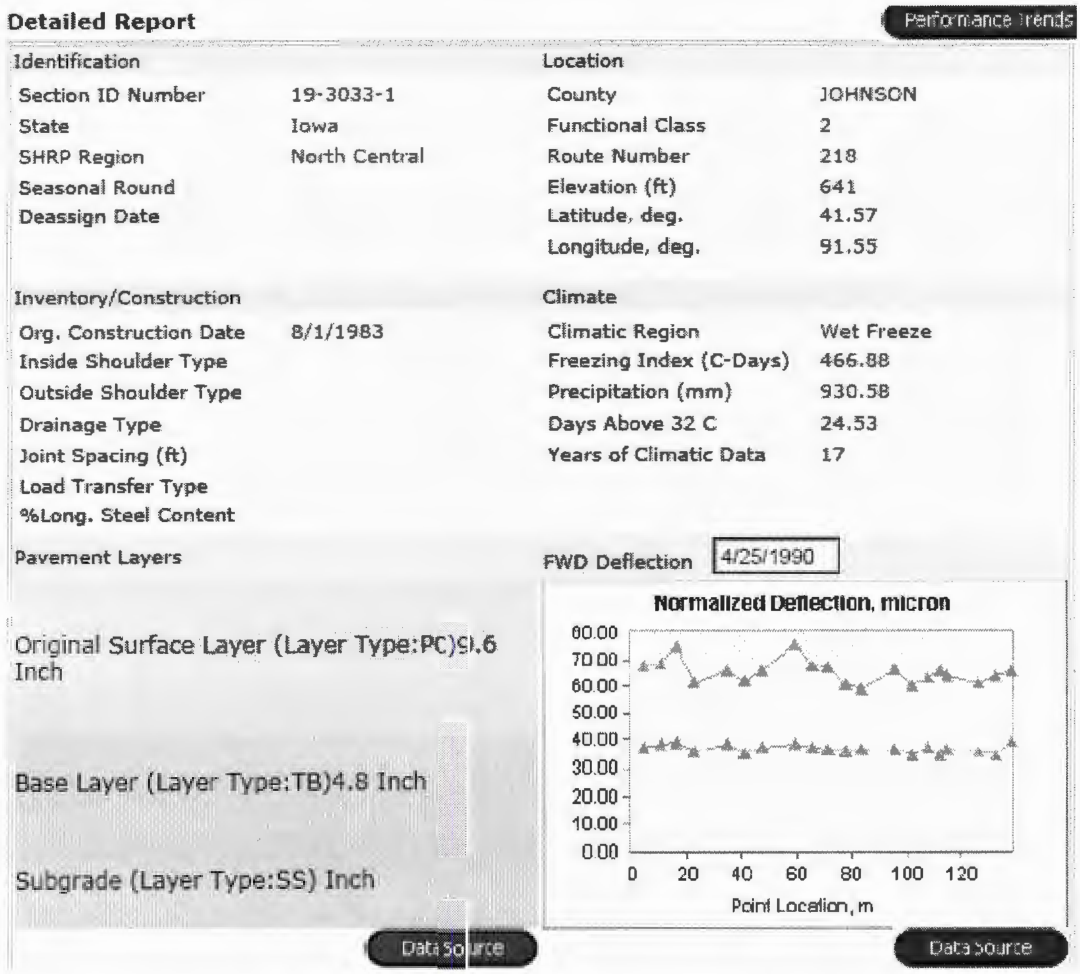

Figure 4.2 PCC-1: LTTP information [4.1]

4.2.2 PCC-2

PCC-2, located on US-20 near Hamilton County, Iowa, was constructed in 1968. The test section was west-bound in the north central LTPP SHRP region, and designated between 149.5 and 153.47 miles of US-20. A complete listing of information obtained is summarized in the Tables 4.6, 4.7, 4.8 and 4.9; and Figure 4.3 shows the summary of general information from LTPP data.

4.2.2.1 Traffic

In 1968, the pavement carried a two-way average daily traffic (ADT) of 3,160 vehicles per day, including heavy trucks. In 2002, it was 5,610 vehicles per day, including 840 vehicles of truck traffic.

4.2.2.2 Climate

This section of US-20 is located in the wet-freeze environmental region. This area has a freezing index of 763.69, and receives 861.74 mm of rainfall annually. The latitude and longitudes are given as 42.46 and 93.59 degrees respectively.

4.2.2.3 Structure

The pavement is a 10-inch JPCP with 15 ft joints. The slab rests on 4 inch (it is mentioned as 3.2 in granular base in LTPP database) granular sub-base course. The subgrade of the site is AASHTO A-6 (7) to A-6 (10) material and it is noted that the soil is glacial till soil. The modulus of subgrade (k) of this section is taken as 150 pcf in the project files.

Table 4.6 PCC-2: Location information

County Name:	Hamilton County (40)
LTPP Section ID Number:	19-3055-1
LTPP SHRP Region:	North Central
Functional Class:	2
Route Number:	20
Elevation (ft):	1186
Latitude (deg.):	42.46
Longitude (deg.)	93.59
Milepost:	149.5-153.47

Table 4.7 PCC-2: Pavement information

Construction Date:	11/2/1968
Surface Layer:	10 inch PCC
Base Layer:	3.2 inch Granular Base (GB) (4 inch GSB, project file)
Subgrade:	SS (Glacial Till Soils A-6 (7) to A-6 (10)
Subgrade k (pcf):	150
Modulus of Rupture from 3rd point loading (psi):	-

Table 4.8 PCC-2: Climate information

Climatic Region:	Wet Freeze
Freezing Index (C-Days):	763.69
Precipitation (mm):	861.74
Days Above 32 C:	12.24
Years of Climatic Data:	29

Table 4.9 PCC-1: Traffic information

Project No:	F-520-4 (7) –20-40
Direction of Traffic:	West Bound
Used Design Method:	Rigid -PCA
Design Life:	20 years
Designed year:	1965

Table 4.9 Continued

Designed year Traffic (vpd):	3160
Design Life Traffic (vpd):	5610 (@1985)
Design Life Truck Traffic (vpd):	840 (@1985)
Design Life Other Traffic (vpd):	4770 (@1985)
Traffic Vehicle Distribution and ESALs	-

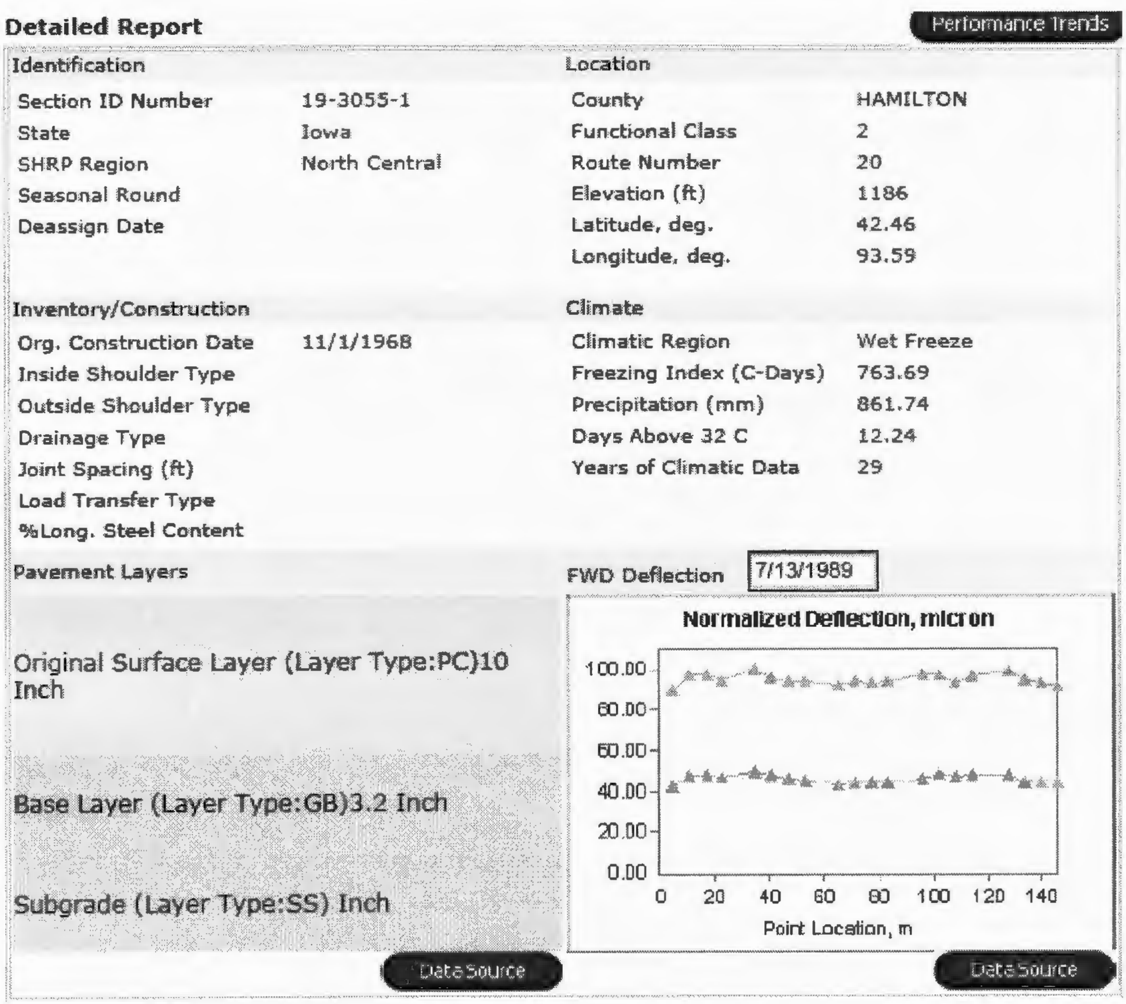

Figure 4.3 PCC-2: LTTP information [4.1]

4.3 MEPDG Analyses of Selected Sites

The data obtained from pavement management information system and LTPP database as described in section 4.2 were introduced to the MEPDG software as inputs. The unknown values are assumed as the default values of the MEPDG, which are nationally calibrated values of the LTPP data sections. The pavement performance values of smoothness were then compared in Figure 4.4 and the results are provided in Table 4.10.

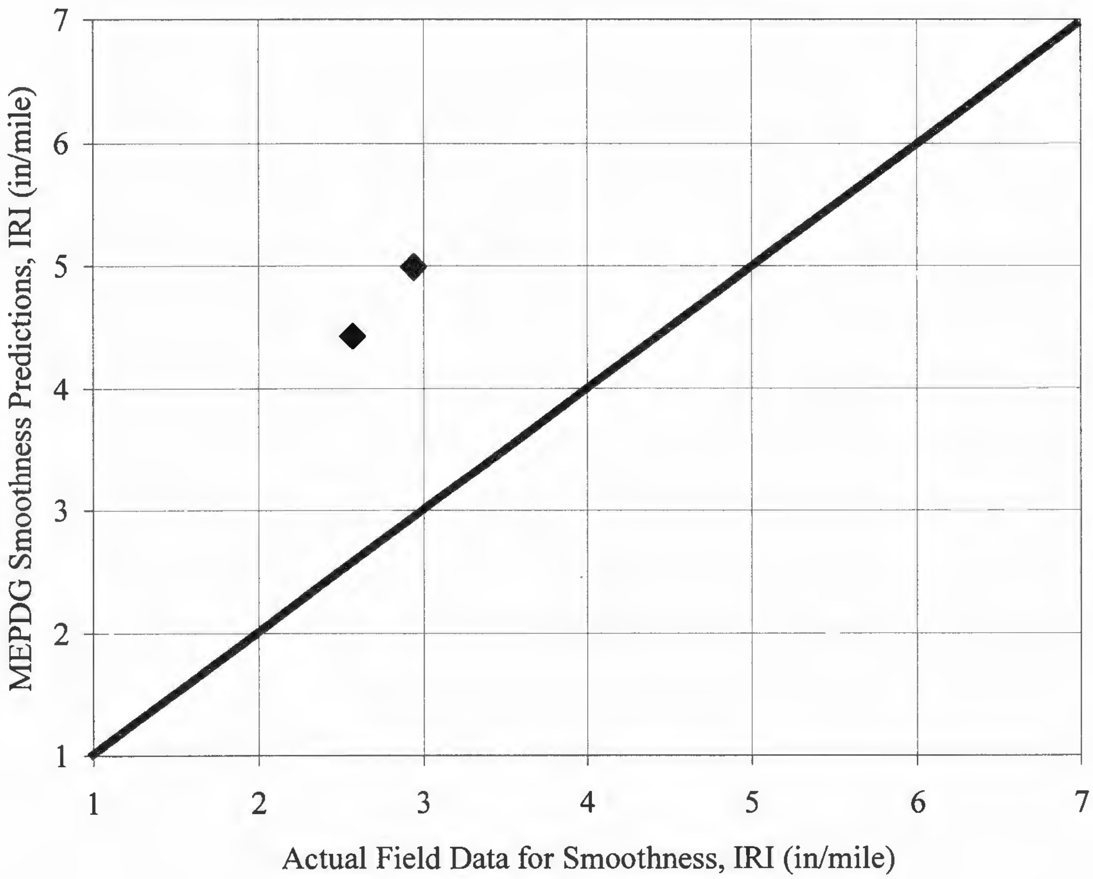

Figure 4.4 Comparison of MEPDG results with PMIS data on pavement smoothness

Table 4.10 Comparison of MEPDG results and PMIS data

	IRI (in/mile)	
	PMIS	**MEPDG**
PCC-1 Johnson(52)	2.57	4.43
PCC-2 Hamilton (40)	2.94	4.99

4.4 Sensitivity Analysis of MEPDG

4.4.1 Overview

Sensitivity analyses were carried out on a representative pavement section to examine the effect of each input or inputs groups of two on pavement performance by using the MEPDG software and the design inputs. The standard input parameters for the representative pavement section of the Iowa highway system was determined by using the inputs similar to the properties of two PCC sections described in section 4.2 and were introduced considering Iowa conditions. A detailed summary of input parameters is given in Table 4.11.

Table 4.11 Summary of standard input parameters for sensitivity analyses

General Information			
Design Life	25 years		
Pavement construction:	May, 2003		
Traffic open:	October, 2003		
Type of design	JPCP		
Performance Criteria		**Limit**	**Reliability**
Initial IRI (in/mi)		63	
Terminal IRI (in/mi)		170	95
Transverse Cracking (% slabs cracked)		15	95
Mean Joint Faulting (in)		0.15	95
Traffic			
Initial two-way AADTT:			6000
Number of lanes in design direction:			2
Percent of trucks in design direction (%):			50
Percent of trucks in design lane (%):			90
Operational speed (mph):			65
Traffic -- General Traffic Inputs			
Mean wheel location (inches from the lane marking):			18
Traffic wander standard deviation (in):			10
Design lane width (ft):			12
Wheelbase Truck Tractor			
	Short	**Medium**	**Long**
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%
Climate			
ICM file:	Ames.icm		
Latitude (degrees. minutes)	41.59		
Longitude (degrees. minutes)	-93.37		
Elevation (ft)	917		
Depth of water table (ft)	2.827		
Structure--Design Features			
Permanent curl/warp effective temperature difference (°F):	-10		
Joint Design			
Joint spacing (ft):	15		
Sealant type:	Liquid		
Dowel diameter (in):	1		
Dowel bar spacing (in):	12		
Edge Support	None		
Long-term LTE(%):	n/a		

Table 4.11 Continued

Widened Slab (ft):	n/a
Base Properties	
Base type:	Granular
Erodibility index:	Erosion Resistant (3)
Base/slab friction coefficient:	0.85
PCC-Base Interface	Bonded
Loss of bond age (months):	60
Structure--ICM Properties	
Surface shortwave absorptivity:	0.85
Drainage Parameters	
Infiltration:	Minor (10%)
Drainage path length (ft):	12
Pavement cross slope (%):	2
Structure--Layers	
Layer 1 - JPCP	
General Properties	
PCC material	JPCP
Layer thickness (in):	10
Unit weight (pcf):	150
Poisson's ratio	0.2
Thermal Properties	
Coefficient of thermal expansion (per F° x 10- 6):	5.5
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28
Mix Properties	
Cement type:	Type I
Cementitious material content (lb/yd^3):	600
Water/cement ratio:	0.42
Aggregate type:	Limestone
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (micro strain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound
Strength Properties	
Input level:	Level 3
28-day PCC modulus of rupture (psi):	690
28-day PCC compressive strength (psi):	n/a

4.4.2 Sensitivity Analysis

The representative pavement section was analyzed with MEPDG software. Then, varying one input parameter within its ranges and holding other parameters constant in the model, several analyses were carried out. Pavement distresses throughout the design life for each input file were plotted. The goal of this analysis was to perform the individual effects of each input parameter on the critical pavement performance using the MEPDG software. It should be noted that the climatic condition reflects Iowa's climate data, and as a variable, the climate input, is considered in or around the Iowa. The chosen weather stations are located in Figure 4.5.

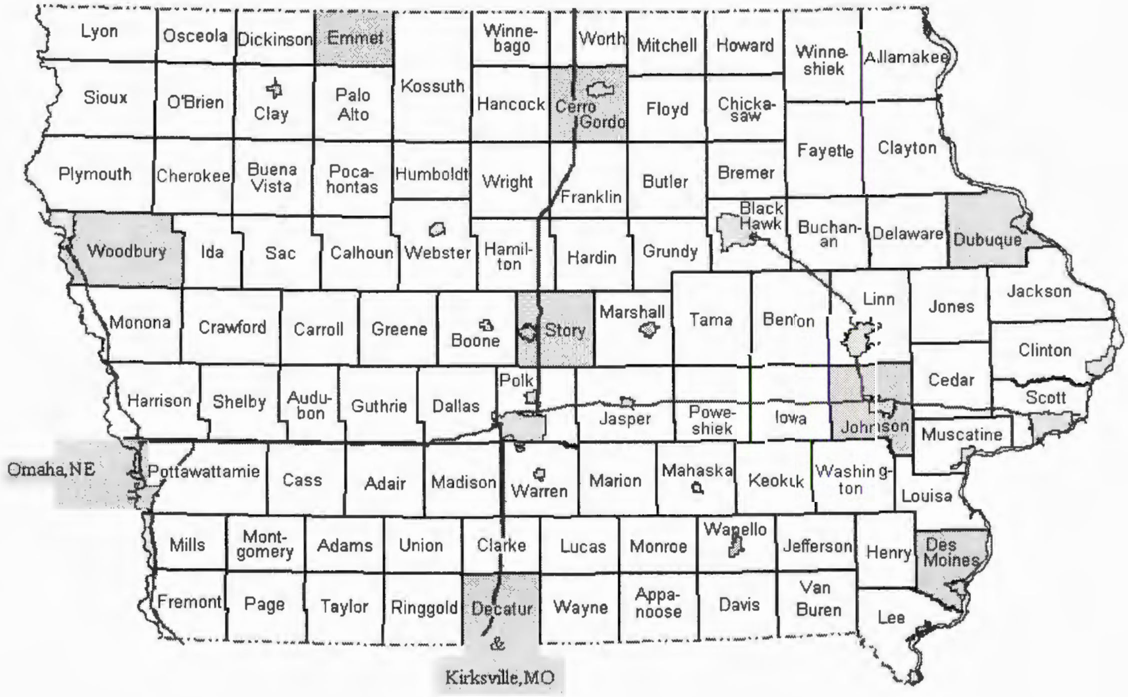

Figure 4.5 The selected climatic locations for sensitivity analysis

The second step was carried out the interaction of input parameters between each other and pavement performance values. The results of the first test (varying one variable) revealed that the standard input parameters established for representative pavement section were corresponding beyond the capacity of pavement performance. Therefore, in some cases the standard input variables were modified to reflect the capacity of pavement performance.

For each input variable, a level of range was defined according to their maximum and minimum values. Moreover, additional values in between minimum and maximum values were considered in order to see the trend of their impact on pavement performance. Several hundreds of graphs were created using the results of MEPDG software. Figure 4.6 through 4.10 are a few examples from such graphs (See Appendix A for all Figures).

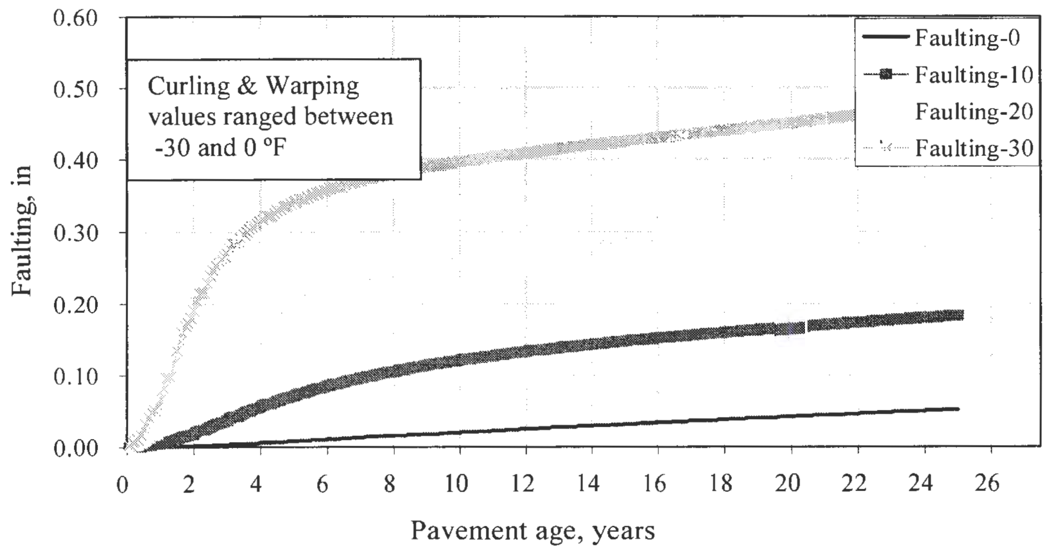

Figure 4.6 Faulting for different curl/warp effective temperature difference (built-in)

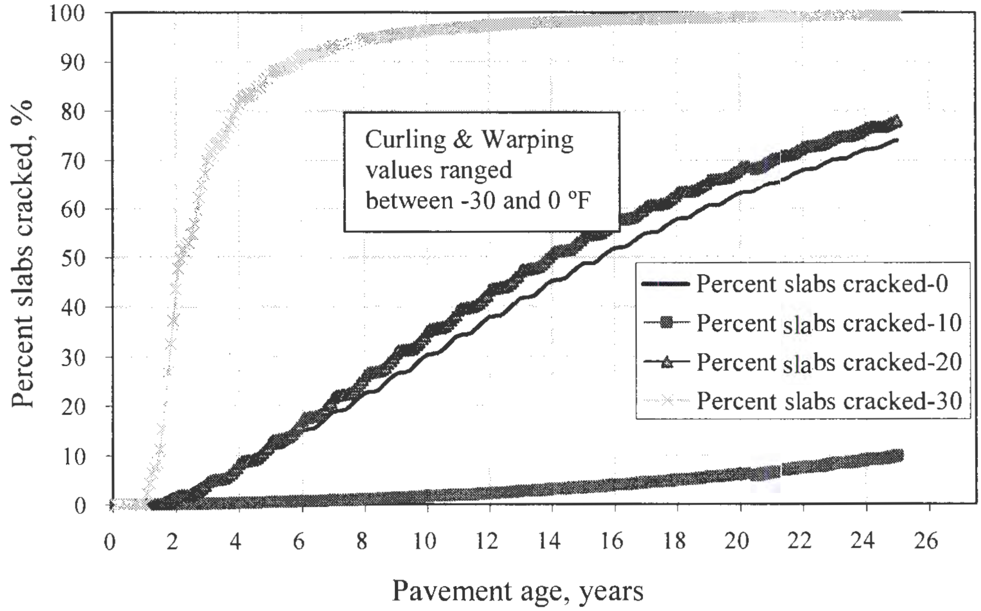

Figure 4.7 Cracking for different curl/warp effective temperature difference (built-in)

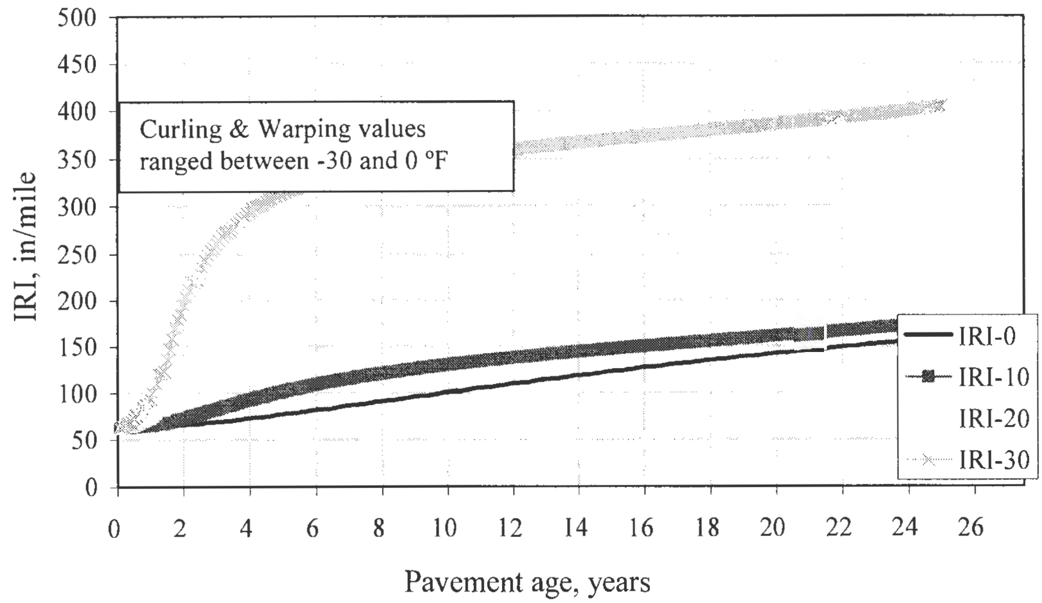

Figure 4.8 IRI for different curl/warp effective temperature difference (built-in)

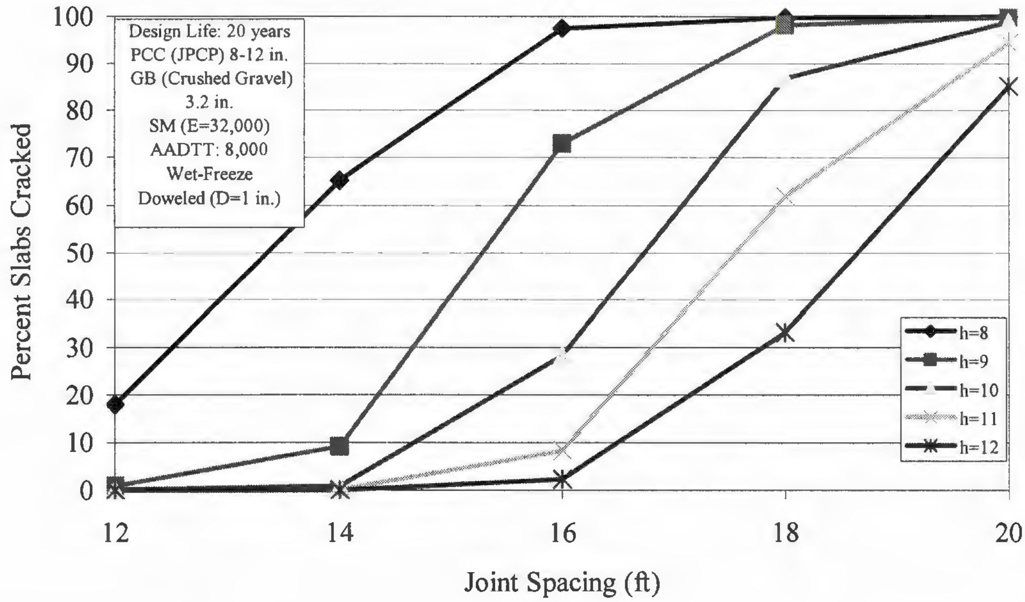

Figure 4.9 Cracking for different joint spacing at different pavement thicknesses

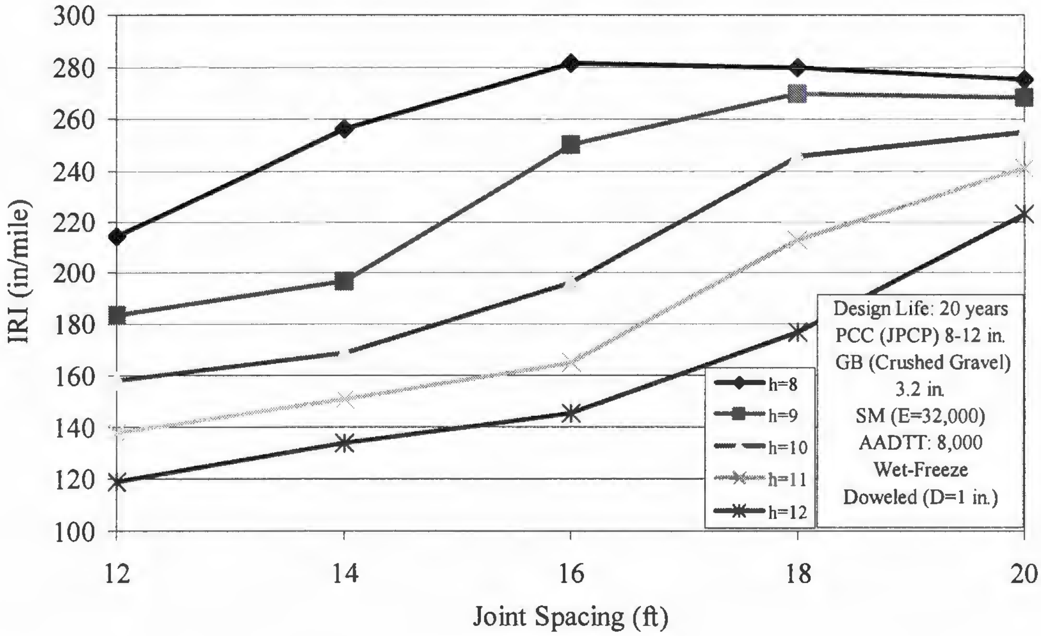

Figure 4.10 Smoothness for different joint spacing at different pavement thicknesses

The obtained plots were visually inspected. The evaluation was made according to the pavement performance value and the amount of change in the pavement performance value due to changing input variable. It can be seen that the results obtained were sensitive in different scales, so the scales shown in Table 4.12 were developed for a better understanding of the effects of input parameters.

Table 4.12 Summary of sensitivity scales

[illegible]	Extreme Sensitivity
VS	Very Sensitive
S	Sensitive
LS	Low Sensitivity
I	Insensitive

Table 4.13 compares the sensitivity values extracted from all of the plots given in Appendix A. Table 4.13 also shows the input scale for sensitivity for each pavement performance and also their hierarchical input level used in the MEPDG software.

Table 4.13 Summary of results of sensitivity analysis for rigid pavements

	JPCP Concrete Material Inputs	Performance Models			Input Levels		
		Faulting	Cracking	Smoothness	Level 1	Level 2	Level 3
Design Features	Curl/Warp Effective Temperature Difference	[redacted]	[redacted]	[redacted]	●	●	●
	Joint Spacing	I/LS	[redacted]	S	●	●	●
	Sealant Type	I	I	I	●	●	●
	Doweled Transverse Joints	[redacted]	I	VS	●	●	●
	Dowel Diameter	I/LS	I	I/LS	●	●	●
	Dowel Spacing	I	I	I	●	●	●
	Edge Support	I	S	LS	●	●	●
	PCC-Base Interface	I	I	I	●	●	●
	Erodibility index	I	I	I	●	●	●
Traffic	AADTT	S/VS	S/LS	S/VS	●	●	●
	Mean Wheel Location	S	S	S/VS	●	●	●
	Traffic Wander	I	I	I	●	●	●
	Design Lane Width	I	I	I	●	●	●
Drainage And Surface Properties	Surface Shortwave Absorptivity	I/LS	LS/S	LS/S	●	●	●
	Infiltration of Surface Water	I	I	I	●	●	●
	Drainage Path Length	I	I	I	●	●	●
	Pavement Cross Slope	I	I	I	●	●	●

Table 4.13 Continued

	JPCP Concrete Material Inputs	Performance Models			Input Levels		
		Faulting	Cracking	Smoothness	Level 1	Level 2	Level 3
PCC General Properties	PCC Layer Thickness	I/LS	[redacted]	S	•	•	•
	Unit Weight	LS	S	I/LS	•	•	•
	Poisson's Ratio	LS	S	S	•	•	•
PCC Thermal Properties	Coefficient of Thermal Expansion	LS/S	[redacted]	[redacted]	•	•	•
	Thermal Conductivity	LS/S	VS/[redacted]	VS	•	•	•
	Heat Capacity	I/LS	I/LS	I	•	•	•
PCC Mix Properties	Cement Type	I/LS	I	I	•	•	•
	Cement Content	LS/S	I	LS/S	•	•	•
	Water/Cement Ratio	LS/S	I	LS/S	•	•	•
	Aggregate Type	I	I	I	•	•	•
	PCC Set (Zero Stress) Temperature	I/LS	I	I/LS	•	•	•
	Ultimate Shrinkage at 40% R.H.	LS	I	LS/I	•	•	•
	Reversible Shrinkage	I	I	I	•	•	•
	Time to Develop 50% of Ultimate Shrinkage	I	I	I	•	•	•
	Curing Method	I/LS	I	I	•	•	•

Table 4.13 Continued

	JPCP Concrete Material Inputs	Performance Models			Input Levels		
		Faulting	Cracking	Smoothness	Level 1	Level 2	Level 3
PCC Strength Properties	28-Day PCC Modulus of Rupture	**LS/I**	[redacted]	**S**			●
	28-Day PCC Compressive Strength	**I**	[redacted]	**S**			●
Unbound Layer Properties	Modulus (Coarse Grained Soils)	**I**	**I**	**I**			●
	Modulus (Fine Grained Soils)	**I**	**I**	**I**			●
	Modulus	**LS/S**	**LS/I**	**S/LS**			●
Climate (in Iowa)	Climatic Data from Different Stations	**LS**	**LS/S**	**LS**	●	●	●

Results for each pavement performance model can be summarized based on faulting, transverse cracking, and smoothness as follows:

4.4.2.1 Summary of Sensitivity Results for Faulting

Faulting is an important pavement performance criterion and has a negative effect on ride quality. It is defined as the differential elevation across the joint and is a result of heavy axle loads, insufficient load transfer between adjacent slabs, free moisture beneath the pavement, and erosion of the supporting base or subgrade material from beneath the slab [4.2]. Usually the approach slab is higher than the leave slab due to pumping, the most common faulting mechanism. Faulting is noticeable when the average faulting in the pavement section reaches about 2.5 mm (0.1 inch). When the average faulting reaches 4 mm (0.15 in), diamond grinding or other rehabilitation measures should be considered [4.3]. Significant joint faulting has a major impact on the life cycle costs of the pavement in terms of rehabilitation and vehicle operating costs.

The following Table 4.14 summarizes the sensitivity scales of the parameters for the faulting performance of JPCP. In the table, the sensitivity of inputs are given under three columns - extreme sensitivity, sensitive to very sensitive, and low sensitive to insensitive.

Table 4.14 Summary of sensitivity level of input parameters for faulting of JPCP

Performance Models	Inputs		
	Extreme Sensitivity	**Sensitive to Very Sensitive**	**Low Sensitive to Insensitive**
Faulting	• Curl/Warp Effective Temperature Difference • Doweled Transverse Joints	• AADTT • Mean Wheel Location • Unbound Layer Modulus • Cement Content • Water/Cement Ratio • Coefficient of Thermal Expansion • Thermal Conductivity	• Sealant Type • Dowel Diameter • Dowel Spacing • PCC-Base Interface • Erodibility Index • Traffic Wander • Design Lane Width • Infiltration of Surface Water • Drainage Path Length • Pavement Cross Slope • Cement Type • Aggregate Type • PCC Set (Zero Stress) Temperature • Ultimate Shrinkage at 40% R.H. • Reversible Shrinkage • Time to Develop 50% of Ultimate Shrinkage • Curing Method • Edge Support • Surface Shortwave Absortivity • Unit Weight • Poisson's Ratio • Climate • PCC Strength • Joint Spacing

4.4.2.2 Summary of Sensitivity Results for Transverse Cracking

Transverse cracking is the key structural failure distress for JPCP. These cracks are usually caused by a combination of heavy load repetitions and stresses due to temperature gradient, moisture gradient, and drying shrinkage [4.4]. As transverse cracking in JPCP increases, further cracking forms. This may lead to the replacement of the whole slab. Slab replacement is costly and can lead to early rehabilitation of the pavement as more occurs. Because transverse cracking is the primary structural design criterion, there should not be many of these occurring in regular projects. However, the AASHTO design guides does not provide a procedure for directly checking a pavement design for transverse cracking, and the guides do not provide adequate recommendations. [4.5]

Table 4.15 summarizes the sensitivity levels of input parameters for the transverse cracking of JPCP. The sensitivity of inputs is summarized under three columns - extreme sensitivity, sensitive to very sensitive and low sensitive to insensitive.

Table 4.15 Summary of sensitivity level of input parameters for transverse cracking of JPCP

Performance Models	Inputs		
	Extreme Sensitivity	**Sensitive to Very Sensitive**	**Low Sensitive to Insensitive**
Cracking	• Curl/Warp Effective Temperature Difference • PCC Thermal Properties (Coefficient of Thermal Expansion, Thermal Conductivity) • PCC Layer Thickness • PCC Strength Properties • Joint Spacing	• Edge Support • Mean Wheel Location • Unit Weight • Poisson's Ratio • Climate • Surface Shortwave Absortivity • AADTT	• Sealant Type • Dowel Diameter • Doweled Transverse Joints • Dowel Spacing • PCC-Base Interface • Erodibility Index • Traffic Wander • Design Lane Width • Infiltration of Surface Water • Drainage Path Length • Pavement Cross Slope • Cement Type • Cement Content • Water/Cement Ratio • Aggregate Type • PCC Set (Zero Stress) Temperature • Ultimate Shrinkage at 40% R.H. • Reversible Shrinkage • Time to Develop 50% of Ultimate Shrinkage • Curing Method • Unbound Layer Modulus • Heat Capacity

4.4.2.3 Summary of Sensitivity Results for Smoothness

Smoothness is an extremely important characteristic of a pavement's performance. Smoothness is also referred to as "roughness". Pavement smoothness greatly affects ride quality, safety, and vehicle operation speed costs which are very important to the traveling public. *Sayers and Gillespie* [4.6-9] define road roughness as the variation in surface elevation that induces traversing vehicles. Roughness is caused by surface irregularities. Surface irregularities either are built into a pavement during construction or develop after construction due to traffic, climatic, and other factors [4.10]. One measure of the pavement roughness provided in the LTPP data base is the international roughness index (IRI), established in 1986 by the World Bank. IRI is calculated from the longitudinal road profile and is reported in units of inches/mile or meters/kilometer. IRI has been shown to correlate with the present serviceability rating (PSR), which is a subjective user rating of the existing ride quality of the pavement [4.11]. The sensitivity results of the MEPDG compare the sensitivity of input parameters that significantly affect JPCP smoothness as measured by IRI. Table 4.16 summarizes the input parameters that affect the smoothness of the JPCP pavement with its sensitivity level.

Table 4.16 Summary of sensitivity level of input parameters for smoothness of JPCP

Performance Models	Inputs		
	Extreme Sensitivity	**Sensitive to Very Sensitive**	**Low Sensitive to Insensitive**
Smoothness	• Curl/Warp Effective Temperature Difference • PCC Thermal Properties (Coefficient of Thermal Expansion, Thermal Conductivity)	• Doweled Transverse Joints • AADTT • Mean Wheel Location • Joint Spacing • PCC Layer Thickness • PCC Strength Properties • Poisson's Ratio • Surface Shortwave Absortivity • Unbound Layer Modulus • Cement Content • Water/Cement Ratio	• Sealant Type • Dowel Diameter • Dowel Spacing • PCC-Base Interface • Erodibility Index • Traffic Wander • Design Lane Width • Infiltration of Surface Water • Drainage Path Length • Pavement Cross Slope • Cement Type • Aggregate Type • PCC Set (Zero Stress) Temperature • Ultimate Shrinkage at 40% R.H. • Reversible Shrinkage • Time to Develop 50% of Ultimate Shrinkage • Curing Method • Edge Support • Climate • Unit Weight

4.5 References

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CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Overview

Mechanistic-Empirical Pavement Design Guide (MEPDG) is based on mechanistic-empirical design procedures and also known as the NCHRP Project 1-37A Mechanistic-Empirical Pavement Design Guide for Design of New and Rehabilitated Pavement Structures. MEPDG includes (1) a guide for mechanistic-empirical design and analysis, (2) companion software with documentation and user manual, and (3) an extensive series of supporting technical documentation. The key improvements that have been included in the MEPDG that make it superior to the 1993 AASHTO Guide are: (1) the use of mechanistic-empirical pavement design procedures, (2) the implementation of performance prediction of transverse cracking, faulting, and smoothness for jointed plain concrete pavements, (3) the addition of climatic inputs, (4) better characterization of traffic loading inputs, (5) more sophisticated structural modeling capabilities, and (6) the ability to model real-world changes in material properties. In short, mechanistic-empirical pavement design procedure is one that uses the principles of both engineering mechanics and field verification to come up with a design process. Mechanistic methods are used to predict pavement responses and their performance is predicted based on performance data collected from "real world" pavements. Due to the complexity of its design procedure, it also has more inputs with its hierarchical approach to the design inputs.

This thesis presents the results of sensitivity investigation on input parameters of rigid modules of the MEPDG. First, a comprehensive literature review addressing the design methods and guidelines of concrete pavements was prepared. Consequently, an overview of the rigid pavement design inputs (traffic, climate, and material inputs) of the MEPDG was completed. Next, the analysis of two selected JPCP sections was performed with the user-friendly MEPDG software. The MEPDG results were compared with available actual pavement field data. Then, using the MEPDG software, the sensitivity of rigid module input parameters of the MEPDG was investigated. Using a sensitivity scale, the effects of inputs parameters on pavement performance were summarized. Conclusions drawn from the study and recommendation for future research are presented below.

5.2 Conclusions

The following conclusions were drawn as a result of the sensitivity analyses described in Chapter 4 (see Table 4.13-16):

✓ The extremely sensitive input parameters for transverse cracking are found as:

- Curl/warp effective temperature difference (built-in)
- Coefficient of thermal expansion
- Thermal conductivity
- PCC layer thickness
- PCC strength properties, and
- Joint spacing

In addition, the sensitive to very sensitive input parameters for transverse cracking are:

- Edge support
- Mean wheel location (traffic wander)
- Unit weight
- Poisson's ratio
- Climate
- Surface shortwave absortivity, and
- Annual average daily truck traffic (AADTT)

Other examined parameters are found as less sensitive to insensitive.

✓ The extremely sensitive input parameters for faulting are:

- Curl/warp effective temperature difference (built-in)
- Doweled transverse joints (load transfer mechanism, doweled or un-doweled)

The sensitive to very sensitive input parameters for faulting are:

- Coefficient of thermal expansion

- Thermal conductivity
- Annual average daily truck traffic (AADTT)
- Mean wheel location (traffic wander)
- Unbound layer modulus
- Cement content, and
- Water to cement ratio

Other examined parameters are found as less sensitive to insensitive.

✓ The extremely sensitive input parameters for smoothness are:

- Curl/warp effective temperature difference
- Coefficient of thermal expansion, and
- Thermal conductivity

Furthermore, the sensitive to very sensitive input parameters for smoothness are:

- Annual average daily truck traffic (AADTT)
- Doweled transverse joints (load transfer mechanism, doweled or un-doweled)
- Mean wheel location (traffic wander)
- Joint spacing
- PCC layer thickness,
- PCC strength properties,
- Poisson's ratio
- Surface shortwave absortivity
- Unbound layer modulus
- Cement content, and

- Water to cement ratio

Other examined parameters are found as less sensitive to insensitive.

✓ The Curl/warp effective temperature difference, coefficient of thermal expansion, and thermal conductivity come out to be the most critical design input parameters that affect each performance criteria. Since these input parameters can not be modified, accurate values should be input into the model. The sensitivity of the model to these parameters is extremely high; therefore, pavement performance outputs can vary significantly. Thus, extreme attention should be given to determine input data for these particular parameters. If necessary, material test(s) should be carried out to determine the magnitude of these parameters. Otherwise the accuracy of the predicted pavement distresses differs significantly.

✓ Among of the extremely sensitive and sensitive to very sensitive parameters, the pavement design engineer can only modify; PCC layer thickness, doweled transverse joints, and joint spacing. PCC strength properties is also modifiable provided that pavement design specifications are met.

✓ For pavement smoothness, comparison of the MEPDG analysis and actual field data of the two selected JPCP sites indicated that the use of MEPDG needs to be calibrated for Iowa suggesting that the accuracy of the actual field data is questionable.

✓ Since the available field data for transverse cracking in pavement management information system are in different units then those used in the MPEDG, it is recommended that the units of MPEDG should be correlated to the actual field data.

5.3 Recommendations

Based on observations made throughout this study, the following recommendations are made:

- ✓ A training program for pavement design engineers with an emphasis on which design input parameters to change or to enter with high precision should be implemented.
- ✓ The existing pavement design guides such as 1993 AASHTO Design Guide do not provide performance prediction of pavements. With the new design approach that includes the use of mechanistic-empirical pavement design procedures and prediction of performance models, in-depth knowledge about use of design inputs is required and establishment of an expert system is recommended. An expert system will help pavement design engineers to determine the critical rigid pavement design inputs that should be modified and not modified for rigid pavement design and the use of correct hierarchical level of each design input.
- ✓ Implementation of laboratory testing, field-testing, and non-destructive deflection testing should be started for all design input parameters. Priority should be given to extremely sensitive input parameters.

5.4 Future Research

- ✓ For local calibration, Iowa DOT should select further sites for different climatic locations, traffic loadings, and material characteristics representing Iowa highway and roads. A detailed comparison on pavement distresses of the MEPDG analysis and the actual field data of these sites should be carried out. It is also of paramount importance to collect detailed accurate field data.

- ✓ The correlation between the PMIS data and the MEPDG performance models should be further investigated for a better comparison of the MEPDG results, such that units should be converted.

APPENDIX A

ACCOMPANYING CD-ROM AND SYSTEM REQUIREMENTS

Appendix A is located in CD-ROM, and contains series of graphs for sensitivity analysis of JPCP design inputs constituting the standard pavement section and data used in the analysis presented in Chapter 4 of the text.

System requirements for CD: IBM PC or 100% compatibles; Windows 95 or higher; 32 MB RAM; hard disk (1 GB minimum); Microsoft Word 2000 or higher.